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Waterways and Harbors Division

PROCEEDINGS OF THE
AMERICAN SOCIETY
OF CIVIL ENGINEERS

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Proceedings of the American Society of Civil Engineers

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BANK PROTECTION ON CENTRAL VALLEY STREAMS

By R. W. Barsdale[1]

SYNOPSIS

Bank protection is an important adjunct to flood control and navigation projects in the Central Valley of California. The corrective measures that have generally proven satisfactory for the various types of channels varying from tidal waters to foothill streams are described.

A brief description of the West's most distinctive topographic feature will be of convenient help in understanding the bank protection problems in the Central Valley region. Fig. 1 presents a general map of the Central Valley. The Valley is approximately 400 miles long and from 30 to 60 miles wide. It is relatively flat, varying from 10 ft below sea level in the delta region to about elevation 500 at its extremities. It is bordered on the east by the Sierra Nevada and Cascade Ranges, and on the west by the California Coast Ranges. It is drained by two main river systems, the Sacramento, having its origin in the mountains to the north, and the San Joaquin rising in the southeast. The Sacramento River flows south along the central axis of the valley, meeting the northward flowing San Joaquin River in a delta area about 30 miles west of Stockton. The joined rivers then flow west to San Francisco Bay and discharge into the Pacific Ocean through the Golden Gate.

Both of these river systems are mature, and before settlement of the valley began in 1850, periodically overflowed their banks into flanking overflow basins inundating wide areas of low lying lands. The settlers, who followed

Note.—Discussion open until April 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. WW 4, November, 1960.

[1] Chf. of Levees and Channel Sect., U. S. Army Engr. Dist., Sacramento, Calif.

the miners, found that wealth could be acquired by means other than the shovel and gold pan, and valuable agricultural products rewarded their efforts, even though crops were sometimes damaged or destroyed by flood. In order to reduce this loss, the settlers began a crude system of levees to afford partial protection from the frequent floods.

FIG. 1.—MAP OF CENTRAL VALLEY

This crude levee system, through local, state, and Federal construction, has expanded during the ensuing 100 yr into a very extensive and complex system of flood control, the scope of which is not widely known. There are about 1,100 miles of both project and non-project levees along the Sacramento River system, 2,400 miles along the San Joaquin River system above the delta, and 1,300 miles in the delta area, or a total of 4,800 miles of levee in the

Central Valley. Of this total, about 33%, or 1,600 miles of levee are a part of federally authorized flood control projects. In addition to these flood control projects, there are two federally authorized ship channel projects, the Stockton Ship Channel, extending 96 miles east from the Golden Gate to Stockton, and the Sacramento River Ship Channel, that will extend 43 miles north from the Stockton Channel in Suisun Bay to the vicinity of Sacramento. The banks and levees of the flood control projects require protection primarily from current erosion whereas those of the ship channels require protection from erosion by waves produced by ship traffic and wind. Thus, it may be seen that bank protection must be an integral part of each type of project. This presents a problem in that the same type of treatment will not be economical and satisfactory for all conditions.

Where flood protection is provided by levees, two basic solutions to the bank erosion problem have been developed. One is to construct the levees on a setback alignment sheltered from waves and current by a strip of bank with the river permitted to meander at will within the overflow area, and the other is to confine the river to a definite narrow channel.

The most important consideration in making this determination is economy. In the Central Valley, and particularly along the downstream reaches of the principal streams, the choice was made by the adjacent landowners and the State of California before the Federal government entered the scene. Most of the flood control levees were initially built immediately adjacent to the stream or, at most, back of a minimum berm. Fig. 2, a view of the Sacramento River, downstream from Sacramento, shows the levees immediately adjacent to the river. Construction of the levees at these locations were due to many reasons. The main levees were constructed by floating clamshell dredges having limited reach. Material for levee construction and repair was readily available from the channel. The ground immediately adjacent to the riverbank between the low water channel and the flanking basins was higher, and less material was required for levee construction. Furthermore, these rimlands were the most valuable; because they were subject to inundation for shorter periods of time, they afforded an area on which houses and improvements would suffer the least damage, and they would support higher income crops without damage from high ground water.

As might be expected, local interests soon recognized the need for bank protection and experimented with many kinds. They were, of course, limited in their efforts by lack of adequate funds and used only readily available materials. Little or no evidence of their effort remains. After authorization of the Sacramento River Flood Control Project by the Federal government, the State of California, as a part of their contribution, undertook bank protection and constructed a few reaches of concrete slab, built-in-place, one or two of articulated concrete mat, and several of dumped rock riprap. These are still in existence (1960) and, in some instances, still provide adequate protection. Subsequently, a modification of the Flood Control Act by Congress made bank protection a responsibility of the Federal government, and little bank protection has been placed by either local interests or the state since that time.

Some of the first bank protection placed by the Sacramento District of the U. S. Army, Corps of Engineers consisted of quarry stone dumped on the sloped underwater bank with fitted, hand placed, quarry stone above the low water line. Because of the high labor costs involved in hand placing the quarry stone, the woven lumber mattress, that was initially developed by the Corps

FIG. 2.—SACRAMENTO RIVER DOWNSTREAM FROM SACRAMENTO SHOWING LEVEE IMMEDIATELY ADJACENT TO RIVER

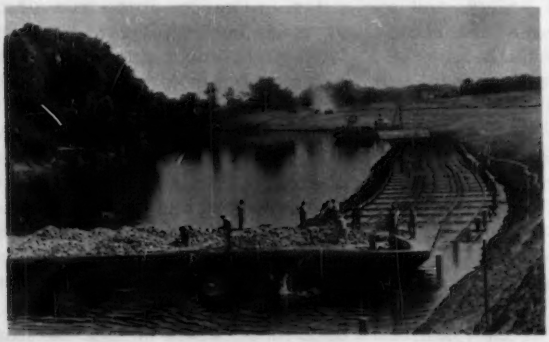

FIG. 3.—COMPLETED MATTRESS BEING BALLASTED SACRAMENTO RIVER ABOVE SACRAMENTO

FIG. 4.—COMMERCIAL ROCK PROCESSING PLANT BAY REGION

of Engineers for use along the Missouri and Mississippi Rivers, was used. These mattresses, of variable width, were laid on the prepared underwater slope from 10 ft riverward of the underwater toe up the slope to the low water line at which they were anchored to piling. Sufficient cobble rock ballast was placed on the mat to sink and hold it in place. The bank above the mat was sloped and received a 12-in. layer of graded cobbles. Fig. 3 shows a completed mattress being ballasted along the Sacramento River above Sacramento.

Due to rising labor costs after World War II, the mat could no longer compete with dumped cobbles from nearby sources. It was also found in some instances that the mattress failed in deep water due to irregularity or the underwater slope causing breakage of the lumber, and failed in shallow water due to floating of the mat after tug boats and barges had dislodged the ballast rock. Quarry rock producers became interested in bank protection, particularily in the delta and the lower San Joaquin and Sacramento Rivers, and asked for the opportunity to bid. The plans and specifications were then changed to provide for alternate bids of either dumped quarry stone or cobbles.

Engineers, of course, recognize that suitable material for bank protection must be durable. In addition, the material should be available in quantity, be economical, reasonably easy to place, and able to resist the applied forces. It is generally agreed that rock is the most common and suitable material meeting these qualifications in the Central Valley.

The Sacramento District is fortunate in having almost unlimited supplies of rock at nearby locations. In the Bay area, hard durable sandstone ledge rock from quarries is being commercially mined and processed for highway use and for breakwater material. Fig. 4 shows the plant that produces most of the quarry rock used in the Sacramento District. The ledge rock is drilled and shot down to the quarry floor on which it is loaded by a power shovel into trucks, hauled to a 6 ft-by 5 ft primary crusher and reduced to a maximum size of 14 in. The crushed material falls onto a conveyor and is carried to the screening plant. All material retained on the 5-in. screen is carried by conveyor and dropped onto 1,200 ton to 1,500 ton barges that are then towed to the site of the work. Here a whirly, or crane, equipped with a clamshell bucket mounted on a barge, picks up the quarry stone and places it on the prepared slope.

At the foothill line along the Sierra Nevada, most of the streams tributary to the Sacramento, Feather, and San Joaquin Rivers have large deposits of cobbles in the form of tailing piles left by the gold dredging operations. Fig. 5 is an aerial view of tailing piles resulting from gold dredging operations. These piles are the source of the cobbles used. The tailing material is run over a grizzly, and all material passing a 12-in. screen and retained on the 1-1/2 in. screen is loaded by clamshell into trucks and hauled to the site. The material is dumped as near to the prepared bank as possible. From here, it is picked up by clamshell and placed on the slope.

Due to the many miles of existing levee within the limits of the authorized projects and to the relatively high cost of bank protection, it has been the practice to place bank protection only at locations in which erosion is active and in which it actually threatens the safety of the authorized levee, or navigation works (or combinations of the two), as opposed to placing it at all locations at which erosion might occur. Most of the bank protection that has been placed by the Federal government is in the delta of the Sacramento and San Joaquin Rivers, and on the lower and middle reaches of the major streams in which the mean channel velocities are less than 10 fps.

Bank protection standards in use in the Sacramento District are the result of experience gained since about 1936. These standards compare very favorably with standards in use by other district offices of the Corps of Engineers. The Chief of Engineers, has suggested criteria for slope protection for dams, levees, and channels. This suggestion is based on investigational projects conducted by various district offices of the Corps of Engineers between 1948 and 1952. The memorandum gave a range of velocities with the corresponding riprap thickness, maximum size, approximate grading, and filter

FIG. 5.—AERIAL VIEW OF TAILING PIPES

thickness for each, and stated that the velocity to be used in selecting riprap values is that measured within 10 ft of the bank rather than the average stream velocity in the channel. Since actual bank velocity data of these kind are rarely available at erosion sites, use of this memorandum requires that correlation be made between the commonly available mean channel velocity and that which may occur 10 ft from the bank. Detailed measurements of extreme velocity distributions against the outside bank of typical bends on the Sacramento and San Joaquin Rivers have been underway since 1956, and preliminary correla-

tion charts are being prepared. After sufficient data have been obtained, final charts will be developed for future use.

Bank erosion, due to current action, occurs most frequently on the outside of the bends and is usually more severe in which the current and thalweg (line of deepest depth) are immediately adjacent to the bank. Unless repaired, the eroded areas increase in length by extending downstream. Thus, it is important that the bank protection be extended downstream far enough to prevent erosion of the bank beyond the end of the rock. This is not necessarily at the end of the curve, but rather well past the point at which the thalweg has left the bank and begun to cross the channel.

Treatment of the underwater toe is of considerable importance in order to prevent undermining and failure. The Sacramento District has developed three standard methods of toe construction. In determining which of the three to use at a specific site, consideration is given to the type and location of the erosion, the current velocity, and the depth of water below the low water line.

The necessity for a filter blanket under the bank protection is also of importance, since removal of the supporting bank material by action of the water can cause the bank protection to settle and sometimes fail. As a general rule, it is the practice to use filters where there is significant tidal action, where there is considerable wave action or where the bank material is cohesionless and without sufficient range of sizes to serve as its own filter.

For ease of reference and due to similarity of the bank protection problems and solutions, the valley has been separated into four general zones or areas, and bank protection on authorized projects in each is discussed separately.

Zone A - The Delta Area.—This includes the Sacramento River downstream from Rio Vista, the Stockton and Sacramento Deep Water Ship Channels, and the numerous sloughs in the Sacramento-San Joaquin Delta. Fig. 6 is a photograph of a Coast and Geodetic Survey (USC & GS) Hydrographic Chart showing the maze of islands and channels in the Delta Area. Delta waterways are in a tidal reach having an average tidal range of about 4 ft. Water levels are not materially affected by floodwaters, and channel velocities do not exceed 3 to 4 fps. Channels vary in width from 500 to as much as 4,500 ft. The area is subjected to strong prevailing winds from the west with velocities up to 20 mph. Soils are predominately peat interspersed with layers of fine sands and silts. The peat varies in thickness from 0 to more than 40 ft. Most of the levees in this area are the accumulated results of more than 50 yrs of clamshell dredging. The levees have an average top elevation of 12.0, Corps of Engineers Datum. Levee builders in this area have learned by experience that an elevation of about 12.0 is the maximum that can be attained, and that the placement of additional material only results in subsidence of the levee with eruption of the natural ground at or near the landside levee toe. The levees usually have 12 ft to 20 ft crowns with 1-on-2 to 1-on-3 side slopes. The islands or polders inside the levees are subsiding at a rate of about 0.3 ft per yr, and the center of many are now more than 10 ft below sea level.

Bank erosion in this zone is caused by wave wash induced by a combination of navigation traffic and wind. A strong wind across a wide section during an extreme high tide (elevation 10.0) has been known to cause waves to break over the top of the levees. Fig. 7 shows a typical delta levee that has been severely eroded by wave action.

Cross sections and soundings through an eroded reach invariably disclose a flat beach on the waterside at about the low tide line. The toe of the caving bank is at or near elevation 5.0 that is about mean tide level. The top

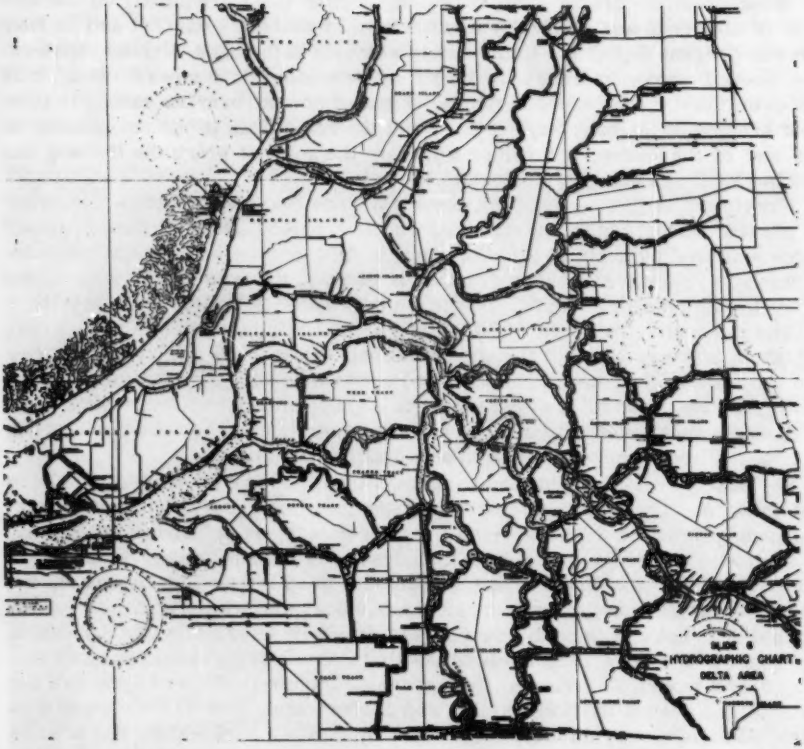

FIG. 6.—HYDROGRAPHIC CHART

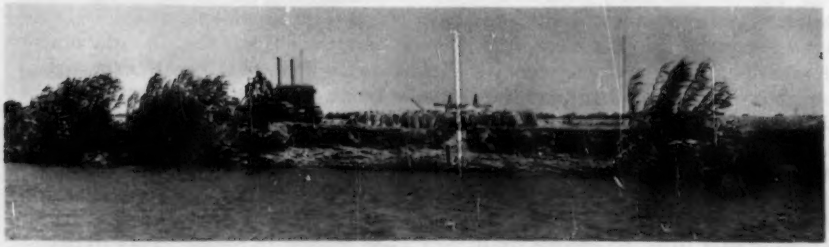

FIG. 7.—TYPICAL DELTA LEVEE SHOWING EROSION BY WAVE ACTION

of the caving bank usually terminates either on the waterside levee slope or in the levee crown. The normal procedure of sloping th riverbank and/or levee from the toe of the caving bank and adding material to the landside of the levee to get the required section is not followed, as the placement of additional material on the unconsolidated peat foundation usually results in serious subsidence and sometimes partial landside failure. Fig. 8 shows a typical section of an eroded delta levee and the method of repair. The current practice is to restore the riverside levee slope with sandy material taken from channel borrow by a floating clamshell dredge. The dressed slope is given a 6 in. blanket of graded filter material followed by a 12 in. thickness of quarry stone protection. Both the filter blanket and the surface course extend from the low water line. If the toe of the erosion is above this line, the filter and surface course are keyed into a shallow trench excavated into the beach or underwater berm. The stone protection is carried up the slope to elevation 10.0, that is 3.0 ft above the elevation of mean high tide. In this zone, the Sacramento District has placed about 52 miles of stone protection since about 1936. The average per foot contract cost of placing bank protection in this zone (including necessary earthwork) based on current price levels is about $15.00 per lineal ft. Two partial bank protection failures have been observed. It was concluded that, in one instance, failure was caused by extreme wave action during an unusually high tide, and the other was caused by abnormal subsidence of the underlying levee.

Zone B - Lower River Area.—This includes the Sacramento River and its distributaries between Sacramento and Rio Vista and the San Joaquin River and its distributaries between Mossdale and Stockton Channel. Waterways in this zone extend from the Delta area, previously described, to the upper limit of the tidal effect. This upper limit is the City of Sacramento along the Sacramento River and Mossdale along the San Joaquin River. Throughout this zone, levees have approximately 1-on-2 riverside and landside slopes and are built immediately on the riverbank or behind minimum berms. The locally built levees were constructed mainly by clamshell dredges; the soils vary from silts to clays. Mean stream velocities in the channels in this area are similar varying from 4 fps to 6 fps. Maintenance practices have been poor with the result that the narrow berms and levees generally support a heavy growth of trees and brush. The 12 ft-crown-width levees along the San Joaquin River and it distributaries in this zone generally have freeboards in excess of 5 ft above the project design flood plane. Those on the Sacramento River, while having a similar freeboard, have crown widths from 20 ft to 30 ft and frequently are occupied by state and county highways. Along the streams in this zone, most of the ranch homes and the highest valued agricultural land are immediately landside of the levee. Bank erosion in this zone is primarily due to current action. Some minor erosion may be caused by waves from a limited number of commercial vessels. Since about 1955 pleasure boats have become so numerous that an increased rate of erosion has been noted along the unprotected banks.

For economic reasons, it is the present practice in this zone to maintain the levee at its existing location. Cross sections and soundings indicate the method of repair. If a berm is in existence, the riverbank is sloped on a 1-on-2 slope from the underwater toe to the berm elevation. Stone protection is then placed from a point 10 ft riverward of the underwater toe up the slope to the berm elevation. Where the berm is narrow, or nonexistent, a 1-on-2 slope is projected riverward from the levee crown. If this slope falls within existing

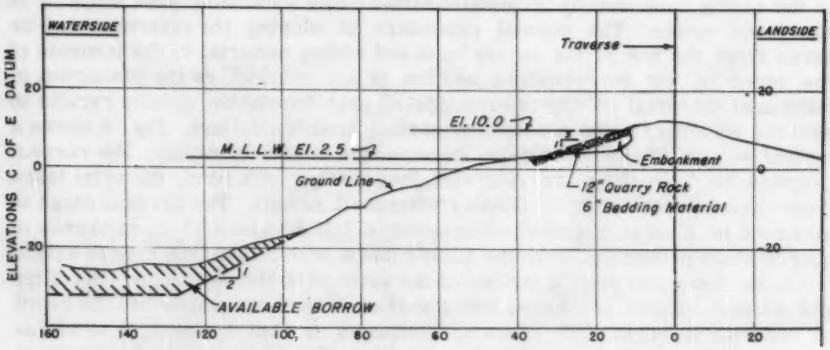

FIG. 8.—TYPICAL SECTION OF AN ERODED DELTA LEVEE AND METHOD OF REPAIR ZONE A

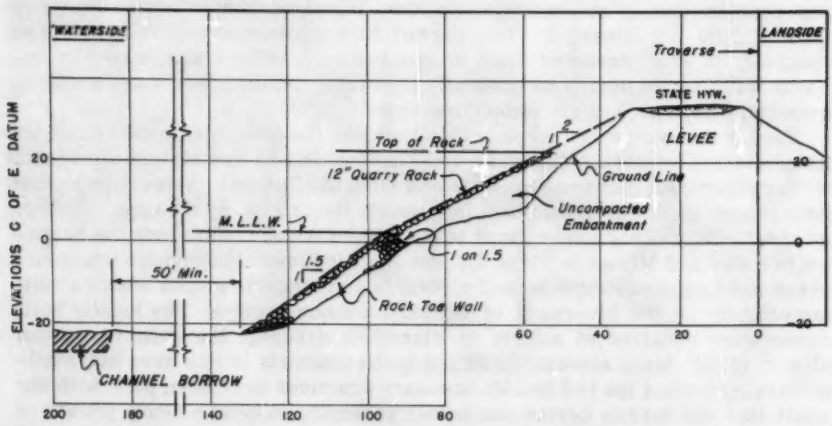

FIG. 9.—TYPICAL SECTION OF ERODED LOWER RIVER LEVEE SHOWING ONE METHOD OF UNDERWATER TOE TREATMENT ZONE B

FIG. 10.—TYPICAL RIVER PLACEMENT OF THE QUARRY STONE PROTECTION ZONE B

ground, the excess material is removed and spoiled and the stone protection is placed from 10 ft beyond the underwater toe up the prepared slope to the elevation of sustained high water during intermediate flood stages. If the slope daylights in the channel, a triangular section rock toe wall with 1-on-1-1/2 side slopes is placed on the streambed with the apex of the wall at the mean low water level and on the projected 1-on-2 slope. Fig. 9 shows a typical section of an eroded levee and one method of underwater toe treatment. Sand from channel borrow is placed behind the toe wall to the water levle. The levee or bank slope, (or a combination of them) is then brought to the desired theoretical section after which stone protection is placed on the prepared slope.

In this zone, the specifications set up alternative bids for the use of either quarry stone or cobbles. However, in the past 10 yr quarry stone, being cheaper, has been placed almost exclusively. Fig. 10 shows typical river placement of the quarry stone protection. In this zone, the Sacramento District has placed about 62 miles of bank protection at an average contract cost of about $30 per lineal foot. There have been no failures of bank protection placed under present standards. There have been, however, 2 or 3 partial failures of bank protection in which the underwater bank was protected by the woven lumber mattress previously described.

Zone C - The Upper River Area.—This includes the Sacramento River between Colusa and Sacramento and the San Joaquin River between the Stanislaus River and Mossdale. Generally, the levees in this zone are separated from the channel by berms varying from 40 ft to about 200 ft in width. The levees, that average about 20 ft in height have at least 20 ft crown widths, 1-on-2 landside and 1-on-3 riverside slopes and have a freeboard of 3 ft or more above the project flood plane. The soils are predominately clays with some interbedded sand stratas. Levees in this zone were initially constructed variously by land equipment, clamshell dredge, and, in one or two instances, by a combination of land equipment and suction dredge. Stream velocities during extreme floods vary from 5 fps to 9 fps. Bank erosion in this zone is due almost entirely to current action, although along the Sacramento River some may be attributed to waves from limited barge traffic and pleasure craft. Fig. 11 shows a typical section of eroded bank in this zone. Water depths along the banks of the Sacramento River vary from 8 ft, during the low water season, to 40 ft, during extreme floods, while on the San Joaquin they vary from 4 ft to 25 ft.

In providing protection in this zone, it is the current practice to slope the riverbank on a 1-on-3 slope from the underwater toe to the berm elevation. Stone protection is then placed from 10 ft channelward of the underwater toe up the slope to the berm elevation. There are some reaches in this zone in which erosion has removed most of the berm. When this condition exists, the bank is cut to a 1-on-3 slope, and the levee is reconstructed on a setback alignment behind a minimum 30 ft berm. Fig. 12 indicates a typical section of a levee in this zone and the method of repair. Specifications in this zone provide for alternate bids of either cobbles or quarry stone, and experience has proven them comparable in price and performiance. If quarry stone is used, it is placed from the river by floating equipment; if cobbles are used, they are truck haudd to the site and placed by land equipment.

Both cobbles and quarry stone have proven satisfactory in this reach, and there have been no failures at locations at which current standards have been followed. However, a woven lumber mattress failed at a Sacramento River site, whereas a partial failure of cobble bank protection was experienced on the San Joaquin River in which the cobbles had been placed on a 1-on-2 slope.

Fig. 13 shows a completed section of bank protection above Sacramento. The Sacramento District has placed 43 miles of bank protection in this zone at an average contract cost of about $50 per lineal foot.

Zone D - Tributary Area.—This includes the main Sacramento River upstream from Colusa, the Feather River, the San Joaquin River above the Stanislaus River, and the tributaries to all of these streams. The Sacramento District has had only limited bank protection experience in this zone. Along the upper Sacramento, San Joaquin, and Feather Rivers, the channel meanders between existing 12 ft to 20 ft crown width levees that are from 1,000 ft to more than 3,000 ft apart. Thus, bank protection is usually not necessary to protect the flood control works. On the tributary streams, however, the levees are closer together, and bank protection is often necessary. Federal levee

FIG. 11.—VIEW OF TYPICAL ERODED BANK IN UPPER RIVER AREA ZONE C

construction along the tributary streams has been underway for about the past three years. The 12 ft crown width levees usually have 1-on-3 riverside and 1-on-2 landside slopes, average 8 ft in height, and provide a three foot freeboard over the project design flood plane. Soils are interbedded layers of clays, silts, sands, and gravels. Channel velocities during project design floods range from 10 fps to more than 12 fps, whereas overbank velocities average 5 fps. Bank protection on the tributary streams is normally required on the outside of curves, through bridges, and adjacent to irrigation and drainage stractures.

The riverbank is sloped on a 1-on-3 slope from a point 5 ft below the present stream thalweg to the berm elevation. Cobbles are usually available near-

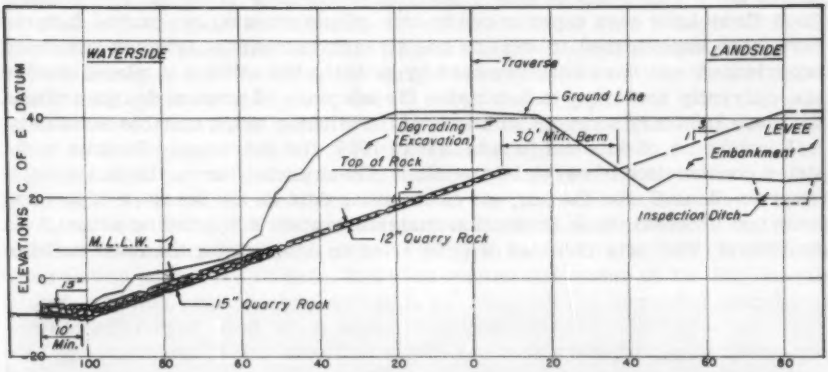

FIG. 12.—TYPICAL SECTION OF ERODED LEVEE ON UPPER RIVER AREA AND METHOD OF REPAIR ZONE C

FIG. 13.—COMPLETED BANK PROTECTION ABOVE SACRAMENTO ZONE C

by and have been specified in past construction. The Sacramento District has placed about 14.0 miles of bank protection in this zone. Although only medium flood flows have been experienced to date on one stream, two partial failures have been experienced to date on one stream, two partial failures have been experienced and have been repaired by grouting the cobbles in place. Studies are currently underway to determine the adequacy of present design criteria for these tributary streams with a view to developing more suitable standards.

It might be of interest to note that in 1959, the Sacramento District completed construction of seven test sections of bank protection on the San Joaquin River in Zone C, for the purpose of obtaining data on the behavior of several kinds of untested bank protection materials when subjected to actual field conditions. The data obtained will be used in determining the most suitable

FIG. 14.—TEST SECTION ON JOAQUIN RIVER UNCOMPACTED ASPHALT BANK PROTECTION ZONE D

and economical available materials on future work along the San Joaquin River as well as on other bank protection projects within the Sacramento District. The sites chosen for the tests are located on the left bank of the San Joaquin River just upstream from Mossdale; they are all on the outside of bends where erosion has been active. Mean channel velocities during the project design flood of 52,000 cfs are expected to reach 7.0 fps. The underlying soils consist predominatly of stratified layers of silty sand and sandy silt with an interbed of sandy clays. At each location, the levee was set back to provide at least a minimum berm of 30 ft between the top of the sloped riverbank and the river-

ward toe of the levee. The natural riverbank was excavated on a 1-on-3 slope from the bed of the river to the berm level at natural gorund.

Material being tested includes (1) graded cobbles on a filter blanket, (2) graded cobbles on natural ground, (3) bank run gravels, (4) processed gravels, (5) bank run dredge tailings, (6) compacted asphalt on a filter blanket above water with quarry stone below, and (7) uncompacted asphalt on natural ground above water with stone below. Fig. 14 shows the completed uncompacted asphalt bank protection. Initial channel cross sections have been secured at each site and will be retaken after major floods. Bank velocity measurements by current meter will be obtained 10 ft from the bank at each site over a range of flows between 10,000 cfs, and the project design flow. At the time that these measurements are taken, the mean velocity in the section will be obtained for correlation purposes. To date, there has been no high water on the San Joaquin River, and the sections are untested. All sites will be inspected annually as well as after reach high water stage when flows exceed 10,000 cfs.

One important factor affecting completed bank protection is maintenance. After a job is completed, local interests usually take the attitude that a permanent solution to the problem has been provided. Whether this is due to the excellence of design and construction or ignorance of the forces of nature, is a matter for conjecture. It is the experinece in the Central Valley that bank protection maintenance is hard to "sell" local interests. It is recognized that the problem is one of education, and we are increasing our efforts in that direction.

Local interests recognize the need for and the importance of bank protection in the Central Valley, and particularly along the Sacramento River. Through Congressional action in 1958, they secured authorization of about 6 miles of bank protection in the upper Sacramento River, upstream from the flood control project, at an estimated cost of $1,500,000. Design funds are expected to be available in July, 1960, and construction will be initiated as soon as local interest requirements have been met. In addition, at the request of Congress, the Corps of Engineers, 1959, completed a survey report on bank protection needs within the limits of the Sacramento River Flood Control Project. If the work is authorized by Congress, as recommended by the Chief of Engineers and the Board of Rivers and Harbors, 80 miles of bank protection will be placed in the next ten years at an overall estimated cost of $22,000,000. These 80 miles, when added to the 171 miles of completed bank protection, will result in about 16% of the banks of authorized projects being protected. It is probable that, as the Central Valley develops, further authorizations for additional bank protection will be forthcoming until, eventually, most of the banks of the Sacramento and San Joaquin Rivers will be protected, not only to prevent damage to the flood control works, but to also save the loss of valuable land.

Journal of the
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RUSSIAN RIVER CHANNEL WORKS[a]

By Israel H. Steinberg,[1] F. ASCE

SYNOPSIS

The paper presents evaluations of channel stabilization works constructed in 1956 by the U. S. Army Corps of Engineers, San Francisco District, along a 4-mile test reach of Russian River, Calif. From observations of this work, tentative conclusions are reached that the works performed as planned and could be adapted to other reaches of Russian River.

INTRODUCTION

In the fall of 1956 the San Francisco District, U. S. Army Corps of Engineers, initiated construction of the first phase of a channel stabilization program having for its aim the alleviation of channel meandering and bank cutting along an approximate 100-mile reach of the Russian River. The initial work was constructed at a cost of about $200,000, and covered a 4-mile reach upstream from the town of Geyserville, which is about 80 miles north of San Francisco along Highway U. S. 101. The work was accomplished as a part of the project authorized by Congress in 1950 in accordance with House Document No. 585, 81st Congress, 2d Session. Another important phase of the authorized project is a multiple-purpose flood control and water conservation reservoir on the East Fork of the Russian River, which is about 45 river miles upstream from Geyserville. This reservoir has been completed and is in full operation. Although the reservoir project will afford a high degree of flow

Note.—Discussion open until April 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. WW 4, November, 1960.

[a] Publication authorized by the Chf. of Engrs., Dept. of the Army.

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reduction in the main Russian River immediately below the mouth of East Fork, additional reservoirs and other measures will be required to eliminate the flooding from the Geyserville area and from the downstream reaches.

STATEMENT OF GENERAL PROBLEM

Bank erosion with accompanying loss of valuable lands in alluvial streams has, for some time, been recognized as a major problem for which many solutions have been sought. The problem is becoming more severe with passage of time due to more extensive developments taking place throughout many parts of the country. This is particularly true in California where the population has increased from 6,900,000 in 1940 to 10,500,000 in 1950, and to an estimated 15,500,000 in 1960. With land at a premium, the channels are being encroached upon to a greater extent, thereby aggravating the problem and, in many cases, the solution as well. Not only is this condition being experienced in areas undergoing urban development, but also it is equally true of practices in agricultural areas.

OBJECTIVE OF INITIAL CONSTRUCTION

The principal objective of the initial works constructed in the Geyserville area is to serve as a guide for remedial measures to be considered for the remainder of the river. The general types of protective measures had been selected in the project document. Some of these types have been constructed and have proven successful in other areas in this country. There was some question, however, regarding their performance on a stream having the flow characteristics of the Russian River. Therefore, the initial works were constructed as a test reach to observe and evaluate their performance. These observations have been made by office personnel on the average of two to three times a year. Any conditions of significance have been noted, evaluations made, and repetitive photographs taken. The information has been compiled in the form of preliminary progress reports. When sufficient observations have been made to permit the evaluation of the works with confidence, a formal report will be prepared that will serve as the basis for types and designs of bank protection measures to be constructed in other reaches of the Russian River.

DESCRIPTION OF BASIN

An understanding of the performance of the channel works requires some knowledge, also, of the characteristics of the stream for which they are designed. A brief discussion of the Russian River is presented, therefore, as a background to assist in the evaluation of the problem.

The Russian River is one of the northern California coastal streams having its origin in the Coast Range Mountains (Fig. 1). It empties into the Pacific Ocean at Jenner, which is about 60 miles northwest from San Francisco. Most of its 1,485-sq mile drainage area is mountainous, although there are a number of valleys along the lower reaches of the tributaries and along the main river where the important economic developments are located. One of these is Alexander Valley where the initial channel works have been constructed. This valley is devoted primarily to agricultural activities with the planting of prune and pear orchards the principal type of development.

The soils in the Geyserville area are comprised of a layer of tan-brown silty sand topsoil averaging about 5 ft in depth and reaching a maximum of 15 ft in some limited areas. The formations beneath this top layer consist of fine and coarse sands, and of gravels which range from 1 in. to 5 in. in diameter. The channel itself is comprised of this latter type of formation. Some reaches of the river have been developed as commercial sources of sand and gravel.

The low water channel in the Geyserville area averages 150 ft in width and meanders between main banks that are more than a half mile apart at some localities. Banks are from 10 ft to 15 ft high. The low water channel is quite unstable, as is typical of a meandering stream, and has changed its course on numerous occasions in the past. This shifting of the channel often creates new

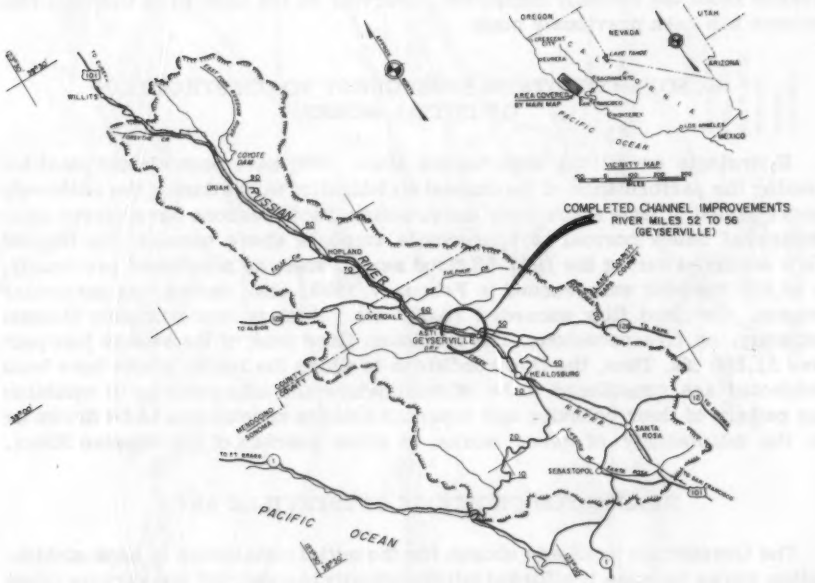

FIG. 1

reaches of the river subject to bank erosion such as occurred following the December, 1955, flood. In the same manner, banks formerly being eroded may become stabilized.

The drainage area of the Russian River above Geyserville is 656 sq miles. Average annual rainfall varies from 38 in. in the valley to 80 in. in the mountains and averages about 43 in. As is typical of California climate, rainfall is seasonal with approximately 80% occurring during the months of November through April. Snowfall is negligible and has no appreciable influence on runoff. Floods are caused entirely by rainfall, usually the result of high intense storms having a duration of two or more days.

Because of the steep terrain and slopes, runoff readily responds to the rainfall and the river will rise from a small base flow to its peak in less than a day. It will recede almost as rapidly. Thus, the river is seldom out of its banks more than a day or so during most floods. The maximum flood flow above Geyserville is estimated as 60,000 cfs which occurred during the December 1955 flood. High flood flows approaching this peak have been experienced also in 1937 and 1940. The maximum flood which occurred since installation of the works was 46,000 cfs in February, 1958. Channel velocities during these flood periods probably exceed 6 ft to 8 ft per second. No actual velocity measurements have been made to date, however, but the estimate is based on consideration of the river slope and channel characteristics and observations at lower flows.

The river flow during the summer and late fall, prior to the rainy season, averages 250 to 300 cfs. This flow is now regulated to a great extent by releases from the recently completed reservoir on the East Fork to which reference has been previously made.

RUNOFF CONDITIONS SUBSEQUENT TO CONSTRUCTION OF INITIAL WORKS

Hydrologic conditions experienced since 1956 have been almost ideal for testing the performance of the channel stabilization works during the relatively short period of time since their construction. Flood seasons have varied from somewhat below normal to appreciable amounts above normal. The highest flow occurred during the 1957-58 flood season when, as mentioned previously, a 46,000 cfs peak was reached in February, 1958. Also, during this particular season, the flood flow exceeded 20,000 cfs, which is approximately channel capacity, on five occasions. The maximum flood peak of the season just past was 31,500 cfs. Thus, the flow conditions to which the initial works have been subjected are considered to be of such nature and magnitude as to establish the pattern of their behavior and to permit definite conclusions to be drawn as to the adaptability of these works to other reaches of the Russian River.

REASON FOR CHOICE OF GEYSERVILLE AREA

The Geyserville area was chosen for the initial installation of bank stabilization works because it afforded full opportunity to construct the various types of work being considered. Other reaches were analyzed also, but were discarded for various reasons.

TYPES OF STABILIZATION WORKS

In general, the stabilization works constructed in the Geyserville reach consist of flexible types of structures that rely on natural forces to assist in their successful performance. The various types of works installed in the test reach consisted of (a) flexible fence, (b) channel training works, (c) barrier jacks, (d) single and double lines of jacks parallel to the bank, (e) wire mesh and gravel blanket, (f) tree pendants, (g) belt planting, and (h) check dams. The locations of these installations are indicated in Fig. 2.

Flexible Fence.—At river mile 56, which is the upstream limit of the initial works, approximately 1,000 ft of the right bank was protected by means of

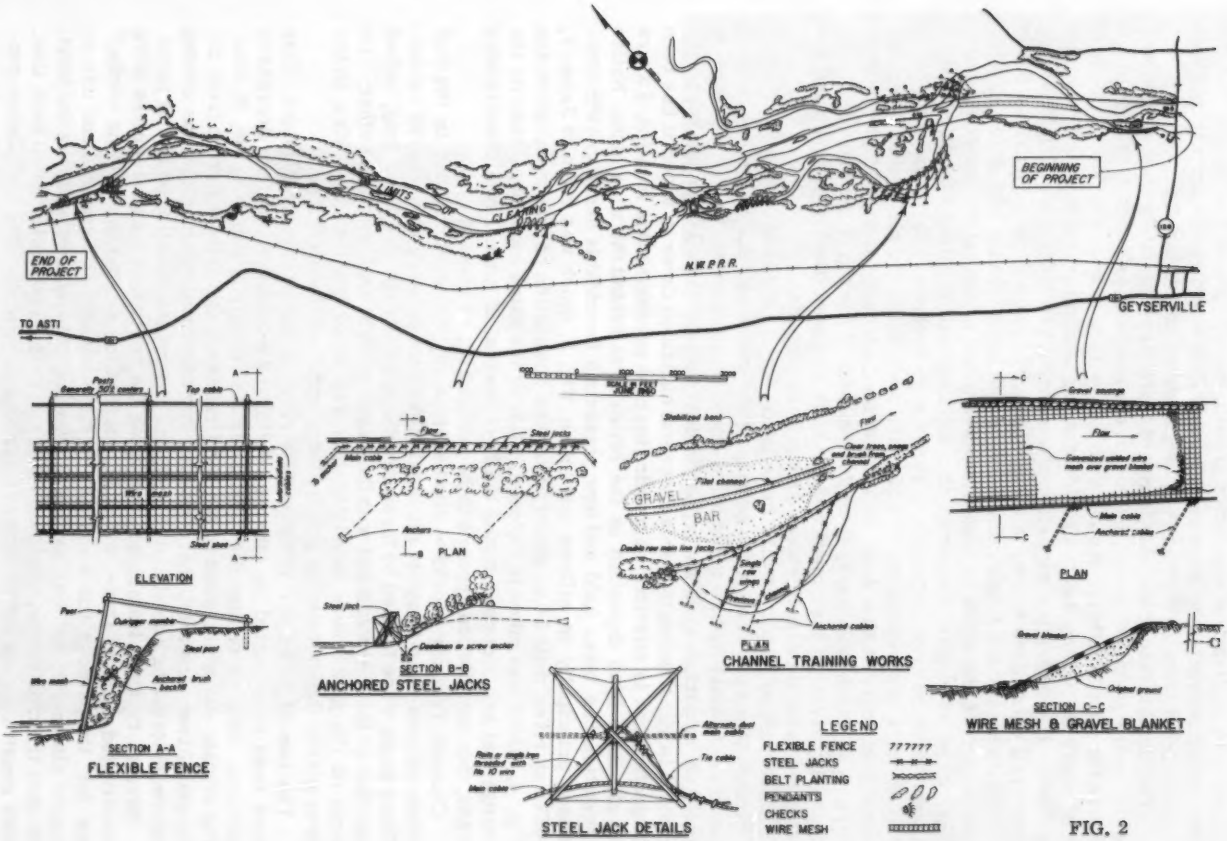
END OF PROJECT
TO ASTI
LIMITS OF CLEARING
N.W.P.R.R.
BEGINNING OF PROJECT
GEYSERVILLE
ELEVATION
PLAN
SECTION A-A
FLEXIBLE FENCE
SECTION B-B
ANCHORED STEEL JACKS
GRAVEL BAR
CHANNEL TRAINING WORKS
PLAN
SECTION C-C
WIRE MESH & GRAVEL BLANKET
STEEL JACK DETAILS
LEGEND
FLEXIBLE FENCE
STEEL JACKS
BELT PLANTING
PENDANTS
CHECKS
WIRE MESH

FIG. 2

a flexible fence type of revetment. Although the top of the bank was well protected with an abundant vegetative growth, the bank itself was unprotected and was being subjected to erosion. Channel flow was directly against this bank which varied from 10 ft to 15 ft in height above the stream bed.

The flexible fence consists of a row of posts, approximately 50 ft on centers, which were installed along the toe of the bank and extended from 5 ft to 8 ft above the top of the bank. A steel shoe 8 ft to 10 ft long was welded to the post 2 ft above its bottom end to provide greater bearing and to prevent excessive settlement. The posts were held in place laterally by bolting them to 30-ft long steel rail outriggers which, in turn, were bolted to steel anchors driven 8 ft to 10 ft into the ground. Single bolts were used to permit hinge action and to allow settlement of the fence in event the stream bed eroded at the toe of the bank. The posts were also connected to main longitudinal cables that were spaced about 6 ft on centers and anchored to deadman anchors. Intermediate anchored cables were also employed to add to the stability of the structure. The posts were then faced with a 4-in. wire mesh such as used for stock fence or hog-wire. The mesh was carried up to the level of the top of the bank only. The space between the wire mesh and the bank was then filled with brush and anchored to prevent displacement during flood periods.

Observations made of this installation (as of 1960) are very encouraging. Although there has been some bulging of the wire mesh due to the pressure of the brush against it, this is not of any serious consequence except for appearance. Of importance, however, is the fact that the installation has been successful in withstanding the force of several high flood flows to which it has been subjected, and is effectively arresting the bank erosion at this location. Layers of silt have been deposited in the brush area behind the wire fence. Native vegetation has taken hold and now presents a formidable dense growth comprised principally of willows and brush. Fig. 3, which was taken in January, 1957, at river mile 56.0, shortly after the completion of this installation and Fig. 4, which was taken in October, 1959, at the same location, illustrate the change that has occurred during the $2\frac{1}{2}$ yr interval of time. From observations made, this installation appears to be performing as planned.

Channel Training Works.—Another type of construction used in the test reach utilizes the concept of the steel jacks held in place by anchored cables. These jacks were formed by three structural steel members, 16 ft long, bolted together at their centers and with their axes at right angles to each other. The arms of the jacks were laced together with steel cable, or with wire, in the three planes thus formed to give added strength to the structure.

The use of jacks for river stabilization is not particularly unique. They have been constructed on other streams and rivers with various degrees of success, such as on the Rio Grande in the vicinity of Albuquerque, N. Mex. The stream characteristics of the Rio Grande are so vastly different from the Russian River, however, that it was felt that additional information was needed before undertaking an extensive program of bank stabilization by use of jacks.

Several types of installations utilizing the concept of jack structures were constructed. One of these, which is referred to as "channel training works," has for its purpose the redirection of the channel at a severe bend into an easier alinement, as well as the alleviation of bank erosion along the bend. Channel training works, covering approximately 2,000 ft of original bank line, were constructed in the vicinity of river mile 53 (Fig. 2). The installation consisted of a double row of steel jacks which were placed across the bend along a selected alinement on $12\frac{1}{2}$-ft centers and held in place by steel cables anchored to the banks. Nine rows of wing jacks were constructed from this main

FIG. 3

FIG. 4

double row to the bank. These wings were more or less uniformly spaced along the main row and radiated at angles approximately normal to the main direction of river flow.

From the first small rise of about 6,000 cfs which occurred in January, 1957, soon after the completion of this work, floating logs and other debris were intercepted by the outer line of jacks and began the formation of a more or less permeable dike as can be seen from Fig. 5. Lesser amounts of debris were caught by the wings, with the amount diminishing progressively down stream. The jack field, and particularly the main line, has continued to collect additional debris with the advent of each flood.

Illustrative of the change that has occurred since the channel training works were first installed is the view (Fig. 6) taken in October, 1959, from approximately the same location as Fig. 5, which was taken following the first river rise after completion. It may be noted that the outer row of the main line of jacks has settled a few feet below its initial level. This is due, no doubt, to the undercutting action of the river in establishing its new alinement. Since no perceptible change has been observed during the past several seasons, it is considered that the condition has now become stabilized and no further settlement of the outer row is expected.

During periods of low and moderate flows the velocities in the jack field are greatly reduced from those in the main river channel. During a field inspection in March, 1960, at which time the river flow was 1,500 cfs, the velocity in the channel near the upper end of the installation, as estimated from a float measurement, was $4\frac{1}{2}$ fps to 5 fps. The velocity in the former channel landward of the outer row of jacks in this vicinity was in the order of $\frac{1}{4}$ fps to $\frac{1}{2}$ fps.

Extensive depositions of silt of variable depths have taken place within the jack field as the result of the slowing down of the velocities. The greatest amount, an estimated 7 ft to 8 ft, has been deposited at the main line of jacks with the depth diminishing progressively toward the bank (Fig. 7). The inner row has become firmly embedded throughout most of its length by the silt deposition, and, at one locality, only the top several feet of the arms of the jacks are still exposed (Fig. 8). Dense growths of willows, brush, and other native vegetation have become well established in this jack field so that they almost hide the structures. It is expected that the growth will eventually spread so as to cover the whole jack field and in this way accelerate the further deposition of silt.

The observations made of the functioning of the channel training works lead to the important conclusion that the success of this type of installation on a stream having the characteristics of the Russian River is dependent upon the effectiveness of the jacks in intercepting sufficient floating debris to create a barrier to the flow. Without this debris, it is doubtful whether the jacks would be of any value since, by themselves, they do not form too great an obstruction to the flow. This can be illustrated by Fig. 9 which shows the channel training works at river mile 52.2 when there was an 8,000 cfs flow in the river. Note the relatively turbulent velocities through the wings where little debris was deposited. Although the velocities in the jack field were about one-half of those in the main channel, they were still appreciable as evidenced by the turbulence at the wing jacks. The jacks themselves give the impression that they have very little effect on impeding the flow of water.

In future installations, consideration should be given to placing some brush along the lines of the jacks at the time of construction. This might prove to be

a flexible fence type of revetment. Although the top of the bank was well protected with an abundant vegetative growth, the bank itself was unprotected and was being subjected to erosion. Channel flow was directly against this bank which varied from 10 ft to 15 ft in height above the stream bed.

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FIG. 5

FIG. 6

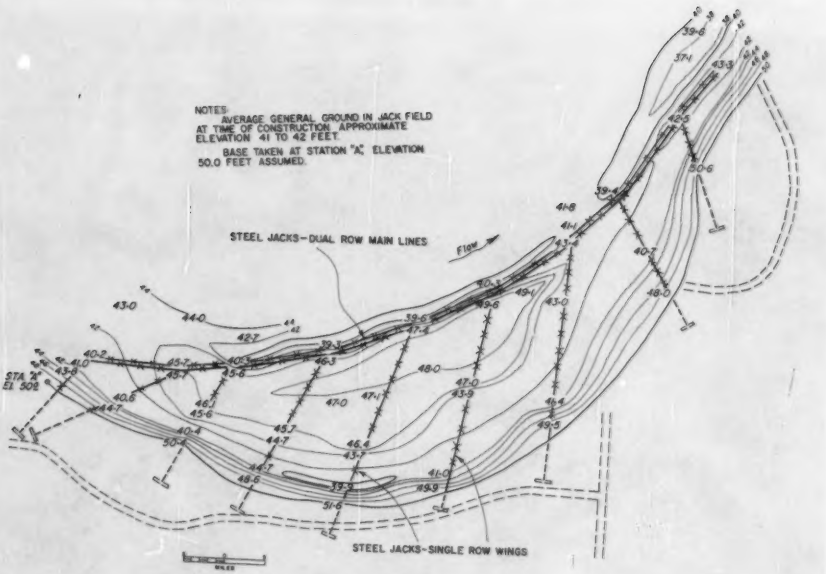
NOTES:
AVERAGE GENERAL GROUND IN JACK FIELD AT TIME OF CONSTRUCTION APPROXIMATE ELEVATION 41 TO 42 FEET.
BASE TAKEN AT STATION "A" ELEVATION 50.0 FEET ASSUMED.
STEEL JACKS-DUAL ROW MAIN LINES
Flow
STA. "A" EL 50.0
STEEL JACKS-SINGLE ROW WINGS

FIG. 7

FIG. 8

FIG. 9

FIG. 10

particularly effective along the wings near the lower end of the installation where opportunity for collection of debris by natural processes is greatly reduced. It is believed this would tend to further reduce velocities in the jack field and encourage the deposition of silt.

Barrier Jacks.—Another type of installation utilizing jack structures was the construction of four rows of jacks across the then existing main channel of the Russian River located on the left side at river mile 52.8. The purpose of these barrier jacks was to effect a diversion of the river flow at low and medium flood discharges into a pilot channel that was constructed as part of this installation. The barrier jacks were initially constructed in straight lines across the channel, as shown in Fig. 10. During a small rise in the river that occurred in January, 1957, shortly after the completion of this installation, the force of the water and floating debris caused everyone of the four lines of cables to snap in two, thus causing the barrier jacks to fail.

Revised designs were prepared, subsequently, for repair of the barrier jacks in which several modifications were incorporated, based on the experience gained from the initial installation. Jack lines were laid out along an arc of a circle having a maximum deviation from the chord equal to one-fifth of the length (Fig. 11). In addition, a heavier cable was used and the method of fastening the jacks to the cables was modified. Also, the spacing between jacks was varied from 20-ft centers at the upstream row to 12.5-ft centers at the downstream row. The purpose of this was to provide a less dense barrier upstream that would permit some of the floating debris to be carried to the downstream lines of jacks.

Since their reconstruction in September, 1957, the barrier jacks have been subjected to the several floods of varying magnitudes mentioned previously. Although some of the jacks have been twisted out of shape, the installation has proven effective in accomplishing its intended purpose. By the end of the flood season following the reconstruction of the barrier jacks, large amounts of debris had been intercepted by the jacks and extensive deposits of silt had taken place in the former channel as seen in Fig. 12. High water flows in subsequent flood seasons added to the amount of deposition, and native vegetation now almost hides the jacks from view (Fig. 13). The low water flow and medium flood flows are now directed into the pilot channel that has increased in width from 15 ft at the time of construction to more than 300 ft at the present time.

Parallel Line Jacks.—Another type of jack installation that was constructed in the Geyserville reach is the one comprised of either a single row, or a double row, of jacks placed adjacent to and parallel to the bank or across the shallow swales. Some of these installations have been very effective. They have collected large amounts of debris, caused extensive deposition of sand and silt, and encouraged the start of native vegetation. Also, in several reaches, the jacks are now almost completely buried by the deposition. This deposition, combined with the dense growth, has nearly obliterated the jack structures from sight. At one installation along the left bank at river mile 53.5, the double row jack line apparently is being flanked somewhat at the upper end of the work. The flow is causing a small channel to form between the line of jacks and the bank. Some remedial measures will need to be taken here to arrest the formation of this channel which might otherwise tend to aggravate the erosion problem in this reach.

Wire Mesh.—A wire mesh and gravel method of bank stabilization was used to check erosion of a 900-ft reach of the right bank near river mile 52. The

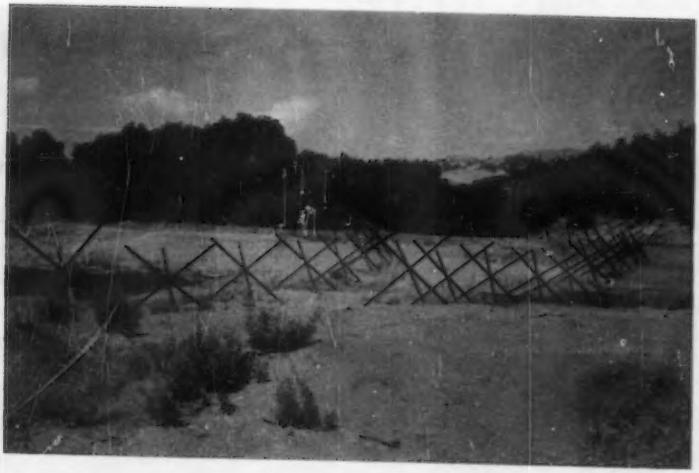

FIG. 11

FIG. 12

FIG. 13

FIG. 14

FIG. 15

bank was first graded to a slope of 1-on-2½ and a 1 ft gravel blanket placed on the slope. Wire mesh was then laid over the blanket and anchored at the top by a longitudinal cable and at the bottom by gravel-filled wire mesh sausages (Fig. 14). In Fig. 14 the gravel sausage, which was subsequently covered with backfill, can be noted.

The observations made following the occurrence of the initial floods revealed that some of the finer materials of the gravel blanket had sifted through the mesh and had been carried away by the flood waters. It was noted, further, that the wire mesh was not in complete contact with the gravel blanket and this was permitting movement of some of the gravels down the slope. This condition was later remedied by driving hooked rods at intervals through the mesh to bring it into closer contact with the gravel blanket. The loss of the finer materials probably can be prevented in future installations by better gradation of the gravel blanket during construction or by using a finer wire mesh. Some silt has been deposited along the slope and top of the bank which encouraged the growth of volunteer grasses and weeds (Fig. 15). The bank appears to be completely stabilized at this location.

Tree Pendants and Belt Planting.—Bank stabilization works included, also, the planting of willow trees or pendants, and willow sprigs at several locations along the river. The pendants consisted of whole willow trees, with root system intact, which were replanted in a trench along the toe of the bank and sloped up the bank with the tops pointing upstream. Both the top and bottom of the pendants were anchored by longitudinal cables. This installation appears to be successful in regard to establishing regrowth. A dense line of willows now (1960) mark the location of the toe of the bank.

The second type of planting consisted of driving 18-in. to 24-in. long willow sprigs into the upper slopes and tops of the banks to form belts of plantings. This is one type of installation, however, which proved to be entirely unsuccessful as all of the sprigs eventually died. Some of the difficulty may have been due to lack of adequate moisture during the summer dry months.

Check Dams.—Although check dams probably do not truly fall into the category of bank revetment works, they are mentioned briefly since they form a part of the installations in the Geyserville area. The check dams were constructed across active swales to arrest their further development and to encourage silt deposition. They were comprised of a gravel fill held in place by wire mesh, steel posts, and wire cable. The center portion of the crest was constructed 6 in. lower than at the ends to minimize the possibility of abutment erosion. This precaution, however, proved to be inadequate, and extensive erosion of the abutments did take place at practically every one of the dams. Remedial measures being planned involve the placing of around the ends of the abutments and extending for distances of about 25 ft upstream and downstream from the dam.

SUMMARY

In summarizing the overall performance of the channel stabilization works, constructed in 1956, in the test reach of the Russian River near Geyserville, the tentative conclusion can be reached that, although several apparent deficiencies in design and construction have developed, the works are functioning (in 1960) substantially as planned. Plans are underway to correct the minor deficiencies and it is expected that this work will be accomplished prior to the end of 1960. Field observations will be continued and the

performance of the work evaluated in a progress report to be prepared by the San Francisco District. This progress report, when approved, will serve as the basis for design of channel stabilization work in the remaining reaches of the Russian River.

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WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

HYDROLOGY AND FLOOD CONTROL FEATURES OF OROVILLE DAM[a]

By Joseph I. Burns,[1] M. ASCE., Kenneth B. Mayo,[2] M. ASCE.,
and William A. Arvola[3]

SYNOPSIS

Oroville Dam, a 735-ft high, zoned earthfill dam, is proposed for construction by the State of California on the Feather River, a major tributary of the Sacramento River. A summary of the hydrologic and meteorologic characteristics of the Feather River Basin and the hydrologic studies pertaining to the design floods for the project are presented. The preliminary spillway design concepts and proposed operating criteria are also discussed.

INTRODUCTION

The State of California has embarked on a vast water development program, termed the California Water Plan, to conserve and use the valuable water resources of the State. The initial unit of the California Water Plan to be built by the State is the Feather River Project. The key unit of the project is a zoned earthfill dam, 735 ft high, to be built on the main stem of the Feather River near the City of Oroville and approximately 70 miles north of sacramento. The reservoir behind the dam will impound 3,523,000 acre-ft of

Note.—Discussion open until April 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. WW 4, November, 1960.

[a] Presented at the June 1960 ASCE Convention in Reno, Nev.

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water. The multi-purpose structure will not only provide conservation storage but will provide urgently needed flood protection to a highly developed flood plain.

Presented in this paper are the hydrologic and meteorologic characteristics of the Feather River Basin, the design floods developed for the project, the preliminary spillway plans for Oroville Dam, and the steps being taken in anticipation of the future operation of the project.

DESCRIPTION OF FEATHER RIVER DRAINAGE SYSTEM

The Feather River drainage system is situated on the western slope of the Sierra Nevada and in the northeastern section of the Sacramento Valley. The drainage system comprises the Feather River Basin, Yuba River, and Bear River watersheds, and the valley flood plain as depicted on Fig. 1. In this paper, the term Feather River drainage system will be used for the entire watershed and the term Feather River Basin will denote the Feather River drainage area above Oroville Dam Site. The drainage areas for the total watershed are as follows:

Drainage Areas	Sq Miles
Feather River at Oroville Dam Site	3,611
Feather River above Confluence of Yuba River	3,978
Yuba River at Mouth	1,350
Bear River at Mouth	550
Feather River at Mouth	5,922

Feather River Basin.—The drainage net of the Feather River Basin, delineated on Fig. 2, comprises three main forks of the Feather River. These three tributaries, North Fork, Middle Fork, and the South Fork, join immediately upstream from the Oroville Dam Site. The North Fork of the Feather River drains 2,090 sq miles or 58 % of the total basin. The eastern part of the basin is drained by the Middle and North Forks, which cut through the abrupt frontal slope of the basin in deep, precipitous canyons. Elevations in the basin range from approximately 200 ft near Oroville to 10,400 ft at Mount Lassen. However, only limited and isolated areas of the Feather River Basin are above the 7,000-ft contour and more than 50% of the area is in the 5,000-ft to 7,000-ft zone. The crest line of the frontal slope, designated on Fig. 2 as top of windward slope, averages 6,000 ft in elevation. The effect of the topography on the natural drainage is to create streams of moderate gradient in the upper portion of the basin. The gradients become quite steep, as high as 150 ft per mile, as the streams flow westerly through the deep, rugged canyons on the frontal slope of the basin.

The natural features of the basin are characterized by extensive areas of merchantable timber land, extremely rough and rocky terrain cut by precipitous canyons, and isolated mountain valleys. At the lower elevations, dense oak groves and low brush are the dominant vegetative cover. Above an elevation of 1,500 ft to 2,000 ft, the oaks merge with coniferous forests. These forests of pine, fir, and cedar are broken by outcrops of granite, deep and rugged canyons, and mountain valleys. The cover of the eastern portion of the watershed is characterized by the growth of less dense coniferous forests

FIG. 1.—FEATHER RIVER DRAINAGE SYSTEM

that encircle the large isolated valleys that are either irrigated or are covered with native grasses and sagebrush.

The climate of the Feather River Basin is characterized by large climatic differences with marked changes in temperature and precipitation within short distances. These changes are caused by the modifying influence of the topography on the movement of air masses. Winter temperatures in the mountain valleys are moderately severe, with minimums below freezing during the period from November to March, inclusive. Summer temperatures are warm throughout the day, with about 95% of possible sunshine, but cool during the nights. Frosts may occur in any month of the year.

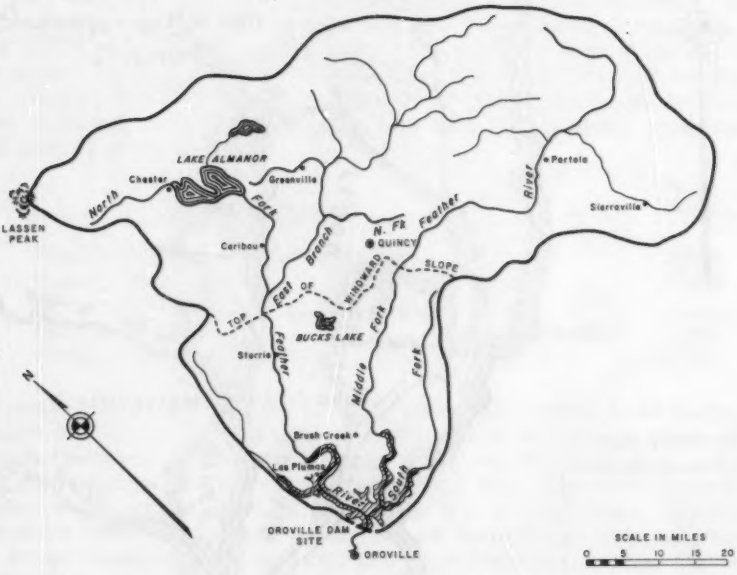

FIG. 2.—FEATHER RIVER BASIN

The abrupt frontal slope of the western face of the watershed has a marked influence on the distribution of precipitation over the basin. Mean annual precipitation amounts vary from approximately 30 in. at Oroville to over 80 in. at the top of the windward slope and then decreases uniformily to approximately 15 in. on the eastern boundaries of the basin. At the higher elevations, the precipitation generally falls as snow and is retained until the spring and summer runoff period.

The seasonal runoff at Oroville Dam Site varies from a maximum of 9,300,000 acre-ft recorded in 1906-07, to a minimum of 1,200,000 acre-ft recorded in 1923-24. The mean seasonal runoff based on 55 yr of record is approximately 4,244,000 acre-ft. The maximum peak flow of 230,000 cfs, estimated from high water marks, occurred in March, 1907. The minimum recorded flow was 300 cfs in November, 1931.

The climate and runoff in the Feather River Basin are recorded by an extensive network of hydrologic stations. Twenty precipitation stations are located with the basin of which nineteen stations have an unbroken record of 10 yr durations or longer. Twelve of the stations are located between 4,000 ft and 5,000 ft elevation. There are nineteen snow courses located within the Feather River Basin that are measured and maintained as part of the California Cooperative Snow Survey program. Nine of these snow courses are located at elevations above 6,000 ft. There are eighteen stream gaging stations located within the basin. Even with this network of stations, many areas with marked changes in climate and runoff are not adequately measured to delineate the considerable variations in these factors.

The North Fork of the Feather River is extensively developed for power generation by the Pacific Gas and Electric Company. The only reservoir that has sufficient storage to exert a major influence on the flood runoff at Oroville Dam Site is Lake Almanor with a total capacity of 1,308,000 acre-ft. This reservoir completely controls approximately 500 sq miles of the North Fork drainage area.

Feather River Flood Plain.—The Feather River flood plain below Oroville Dam Site, depicted on Fig. 1, comprises a total area of 292,000 acres. The flood plain area, as defined, is artificially bounded in certain areas by levees of the Sacramento River Flood Control Project. This total flood plain area is composed of 9,000 acres of urban and sub-urban lands, 245,000 acres of agriculture lands, and 38,000 acres of land that is used for grazing, mining, and game preserves. The 245,000 acres of land used for agriculture consists of 190,000 acres of field and row crops and 55,000 acres of fruit and nuts. The current (1959) value of all lands and improvements in the flood plain is estimated by the U.S. Army Corps of Engineers at about $650,000,000 of which $275,000,000 are urban values and $375,000,000 are rural values. The cities of Marysville and Yuba City, with a total population of 21,000, are the main cities completely within the flood plain. It has been estimated by the Department of Water Resources that the population in the three counties of Butte, Sutter, and Yuba will increase from the 1959 figure of 148,000 persons to approximately 600,000 persons by 2010. Most of the areas of these counties where this growth will occur are within the Feather River flood plain.

The Feather River flood plain area is protected by 205 miles of levees. The levees are a part of the Sacramento River Flood Control Project, which was authorized by Congress in 1917. The design capacities of this levee system are 210,000 cfs above the confluence of the Feather and Yuba Rivers; 300,000 cfs below this confluence; and 320,000 cfs below the confluence of the Feather and Bear Rivers.

Since the turn of the century, major floods have occurred in the Feather River system in 1904, 1907, 1909, 1928, 1937, 1940, 1942, 1950, and 1955. In 1955, 100,000 acres of the flood plain were flooded resulting in the loss of forty lives and a total damage estimated by the Corps of Engineers at $55,581,000. This flood and breaching of project levees resulted in the evacuation of 30,000 people, flooded 3,300 homes, and drowned 6,000 head of stock. Approximately 3,000 acres of peach, walnut, and plum orchards were lost in the flooded area below Yuba City.

METEOROLOGIC CHARACTERISTICS

The storms that produce excessive amounts of rainfall to high elevations in the northern Sierra Nevada with resulting high rates of runoff are character-

ized by strong, sustained southwest winds. The air mass in these storms is identified as modified polar air that has acquired tropical characteristics. All the major flood producing storms in the Feather River drainage system are essentially the same type, differing only in orientation and intensity. During November through April, such storms result from the weakening of the semi-permanent Pacific high pressure center, which greatly influences California's climate, and the strengthening of the Aleutian low pressure center. In this period, storms originating on the Pacific polar front move progressively further south and pass inland through Washington, Oregon, and California. At times, the large-scale atmospheric flow pattern is characterized by a blocking high pressure center at high latitudes that diverts the storms more directly into California from the Pacific and creates a potential flood situation with warm, tropical air masses carried inland by strong winds. When such a storm track over California persists for many weeks, conditions are especially conducive for prolonged rainy periods in the state.

Precipitation in California is largely of orographic origin when strong winds carry moisture-laden air up the slopes of the Coast Range and Sierra Nevada to be released as rain or snow. Precipitation occurring within the storm centers and along the fronts also results from the horizontal convergence of moist air masses. This contribution adds to the orographic component to give the total storm precipitation.

FLOOD RUNOFF CHARACTERISTICS

The flood runoff from the frontal 1,000 sq miles of the Feather River Basin in very rapid. The occurrence of high intensity bursts of precipitation on this windward slope combined with the efficient collection of runoff by the high gradient stream system, concentrates the flow very rapidly and results in the almost "flashy" characteristics of the flood hydrographs at the Oroville Dam Site. These characteristics can best be illustrated by examining the storms and resulting runoff for two recent storm periods.

February 1960 Storm Period.—In the week preceding February 6, 1960, the mid-tropospheric flow pattern was characterized by strong westerly flow over the eastern Pacific with low pressure over the Gulf of Alaska. At the 500-millibar level (mid-troposphere), a series of short-wave troughs moved eastward and entered the Pacific Coast in Oregon, Washington, and British Columbia. On the surface weather chart, the polar front was located approximately 30°N latitude in the Pacific, and on the front a series of waves moved eastward and reached the California-Oregon-Washington coast as occluded fronts.

On February 7, 1960, an active low was located on the polar front about 2,000 nautical miles due west from San Francisco. This low deepened and moved northeastward at a speed of 35 to 40 knots. The cold front associated with this low also moved rapidly eastward and crossed the California coast in the early hours of February 8, 1960. Rain began to fall over the Feather River Basin on the 7th, and the greatest intensity fell during the night of the 7th and early hours of the 8th. During this period, the freezing level was at approximately 8,000 ft. The highest hourly rate of rain as recorded at the Brush Creek precipitation station (Fig. 2) was 0.70 in per hr, and the accumulated rainfall at this station was approximately 9 in. in 26 hr. The cold front passed the Feather River Basin around noon on February 8, and with its passage the heaviest precipitation terminated.

Prior to this storm, there had been only a nominal amount of precipitation on the Feather River Basin resulting in relatively dry antecedent ground conditions. At the beginning of the storm, there was a flow of only 5,000 ocf in the Feather River at Oroville Dam Site. However, the rain associated with the storm resulted in a very rapid rise of the Feather River flow from 5,000 cfs to 133,500 cfs in 24 hr. The hydrograph of runoff at the Oroville Dam Site and the hourly precipitation at Brush Creek precipitation station are depicted on Fig. 3. Note that the peak flow at Oroville occurred almost concurrently with the passage of the cold front and the abrupt cessation of effective rain at Brush Creek. It is apparent that the main contribution to the peak flow at Oroville came from the approximately 1,000 sq miles of the frontal slope area of the basin.

December 1955 Storm Period.—The weather situation in December, 1955, was characterized by an extensive blocking high pressure center located over the Bering Sea which displaced the Pacific polar front further south than normal over the Pacific Ocean. The front extended from a point just north of the Hawaiian Islands northeastward toward northern California. The confluent flow of very moist tropical air and cold air from the north Pacific along the frontal zone produced a very strong front and a very pronounced mid-tropospheric jet stream. Waves formed along the front over the ocean and moved rapidly toward northern and central California. Between the 17th and 24th of December, five waves moved into northern California bringing fove "bursts" of heavy rain lasting generally from 12 hr to 18 hr. The heaviest bursts were on the 19th and 21st. The maximum hourly precipitation rate at Brush Creek was 0.64 in. per hour on the 19th, and 0.85 in. per hour on the 21st. The total rainfall at Brush Creek in the 8-day period amounted to 30 in. The normal precipitation at this station is 69 in. Rainfall was also heavy over other parts of the Feather River Basin and the neighboring Yuba River Basin. The warm air mass carried eastward by the strong winds had a freezing level near 10,000 ft, and therefore, rain fell to high elevations in the mountains. The plot of hourly rainfall at Brush Creek during the period December 18-25, 1955, is shown on Fig. 4.

The resulting runoff, also depicted on Fig. 4, produced three major peaks of 95,000 cfs on December 19, 175,000 cfs on December 22, and 203,000 cfs on December 23. Note that the storm bursts on December 21-22 and December 23 produced approximately twice the runoff per unit precipitation that was produced by the rain of December 19. This can be attributed to the fact that the detention storage and field moisture requirements of the basin were being met and the snow line during this phase of the storm was relatively low. Note that these peak flows occurred approximately 4 hr after the intense bursts of rain at Brush Creek. By the start of the hydrograph rise on December 23, snow below elevation 6,000 ft had effectively disappeared; therefore, there was a negligible contribution from melting snow during the peak runoff. The peak flow of December 23, combined with the record runoff from the Yuba River, resulted in the levee failure below Yuba City on December 24, 1955.

DESIGN FLOODS

The design of the flood control features of the Oroville Dam is of prime importance because of not only the unprecedented height of the zoned type earthfill dam but because of the proximity of the dam to the highly developed Sacramento Valley flood plain.

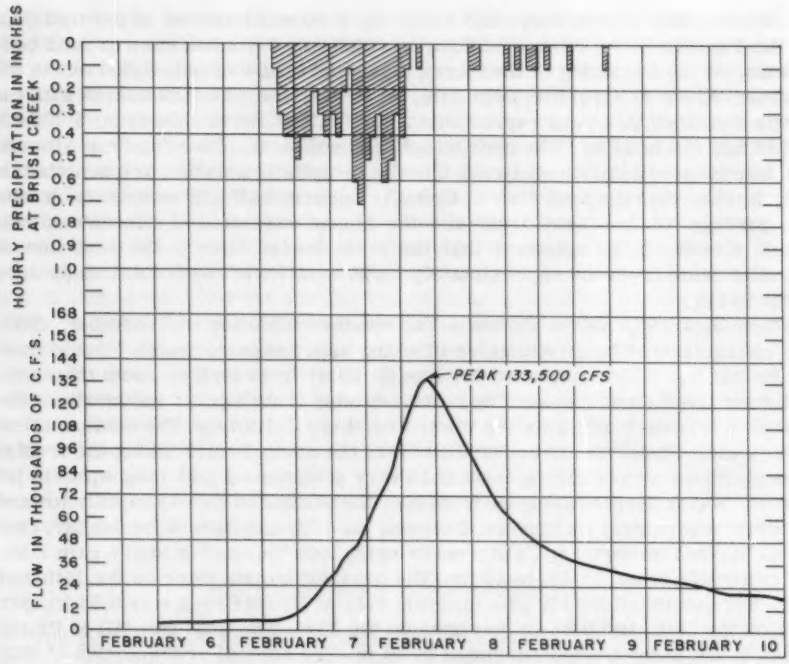

FIG. 3.—PRECIPITATION AT BRUSH CREEK AND HYDROGRAPH AT OROVILLE DAM SITE, FEBRUARY, 1960

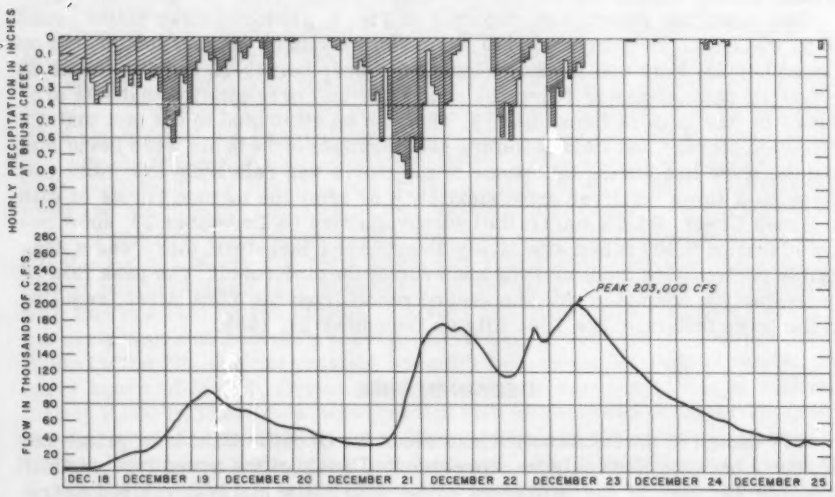

FIG. 4.—PRECIPITATION AT BRUSH CREEK AND HYDROGRAPH AT OROVILLE DAM SITE, DECEMBER, 1955

Probable Maximum Storm Studies by Department of Water Resources.—Because of the tremendous scope of the California Water Plan and the magnitude of many of the individual projects, the Department of Water Resources in 1958 retained Mr. Joseph B. Knox to derive a method for obtaining and evaluating estimates of extreme rates of precipitation on large watersheds in California. The results of these studies have been published by the Department of Water Resources[4].

The principal objective of the study was to develop a technique for estimating maximum accumulated precipitation for the Feather River Basin by applying extreme values of certain meteorological parameters to an atmospheric model. This model has provisions for including both orographic rainfall and rainfall from large-scale synoptic disturbances. These components are combined in an equation that describes the variation of the total precipitation rate with time.

The necessary computations were programmed for an electronic computer. The over-all model was tested by reproduction of precipitation in a 72-hr period during the December 1955 storm in the Feather River Basin.

Orographic Precipitation.—To determine the orographic rate of precipitation, the behavior of the wind field moving over the basin must be known. The influence of the topography upon the winds can be obtained by considering simplified terrain profiles in which the essential features of the mountain barrier are preserved. The simplified terrain profiles were obtained by smoothing the plots of the average terrain in three strips (Fig. 7 and 8), each 16 miles in width and oriented SW-NE across the Feather River Basin (an example of the profiles is depicted on Fig. 6). The initial segment of the simplified profile consists of the sloping temperature inversion layer over the Sacramento Valley. Intervening valleys within the basin are considered to be capped by level inversion surfaces over which the warm air mass is assumed to move horizontally. The inflow winds as a function of height, upwind of the mountain barrier in essentially undisturbed flow, are obtained from meteorological upper-air data. By assuming a distribution of vertical velocity with height, and using the principal of continuity, the horizontal and vertical components of the wind were computed at each 500-m level from the surface to the tropopause (approximately 9,000 m) on verticals spaced at intervals of approximately 2 miles along the profiles. An example of the vertical profiles of the horizontal inflow and outflow winds is shown on Fig. 5.

With the wind field across the basin computed, the trajectories were computed for rain drops of a given size that terminate their earthward fall at the bottom of the verticals in the computational grid. Separate trajectories for rain drops with terminal fall velocities of 4, 6, and 8 m per sec were computed. An example of the rainfall trajectories for rain drops with terminal fall velocity of 4 m per sec for the December 1955 storm, is depicted on Fig. 5 for Profile "A." (The simplified computational terrain profile is also shown on Fig. 5 for Profile "A.") Using the results of the computations of rain drop trajectories, it is possible to compute the orographic rainfall at the base of each vertical in the computational grid, assuming a saturated adiabatic atmosphere of a given moisture content.

Large-Scale Synoptic Rainfall.—The rate of precipitation due to large-scale vertical motion in the atmosphere was computed by applying an assumed vertical velocity (resulting from the dynamics of the storm circulation) to the same

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4 "Procedures for Estimating Maximum Possible Precipitation," by Joseph B. Knox, State of California, Dept. of Water Resources, Bulletin 88, Sacramento, Calif., May, 1960.

air mass used in the orographic computations. Care was taken to insure that the large-scale (termed "frontal") precipitation used could reasonably be expected to occur during the storm-type characterized by extreme orographic precipitation.

Time Distribution of Precipitation Rate.—The time distribution of precipitation rate is based on the fact that moving frontal disturbances, such as occurred during the December 1955 storms, produce a "periodic" variation in the rate of precipitation. The "periodic" rate of precipitation is quite normal as this storm type is marked by a strong southwesterly basic current in which

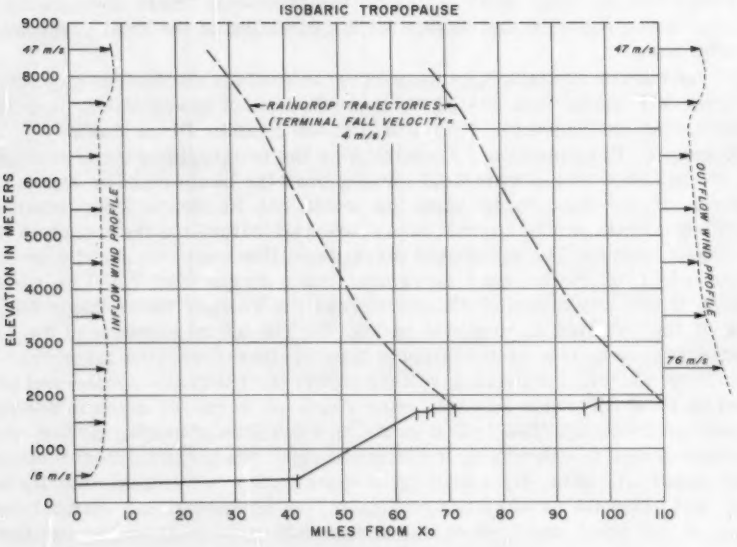

FIG. 5.—INFLOW AND OUTFLOW WIND PROFILES AND SAMPLE RAINDROP TRAJECTORIES, PROFILE "A," DECEMBER 1955 STORM, FEATHER RIVER BASIN

frontal waves propagate. The maximum hourly rates of precipitation are computed from the values obtained in the orographic and frontal precipitation computations. The time-distribution model takes into account the fact that the saturation potential temperature (a measure of the moisture content of the air mass), the horizontal wind, and the frontal component of vertical motion are not independent of time.

The instantaneous rate of precipitation is represented by an idealized sinusoidal function that closely resembles variations in recorded rates of precipitation. Critical periods were computed for this function, with a period of 59 hr being selected as the most critical for a total storm duration of 72 hr.

Nature does not produce precipitation rates that can be represented by a simple sine or cosine curve. However, Mr. Knox has shown that the total precipitation resulting from a complex time distribution of hourly precipitation is less than the total precipitation resulting from the presence of the most

adverse period alone, provided the maximum hourly precipitation in the two cases is the same. For design storms, therefore, the choice of the critical period (for a particular storm duration) represents a reasonable extreme possible in nature.

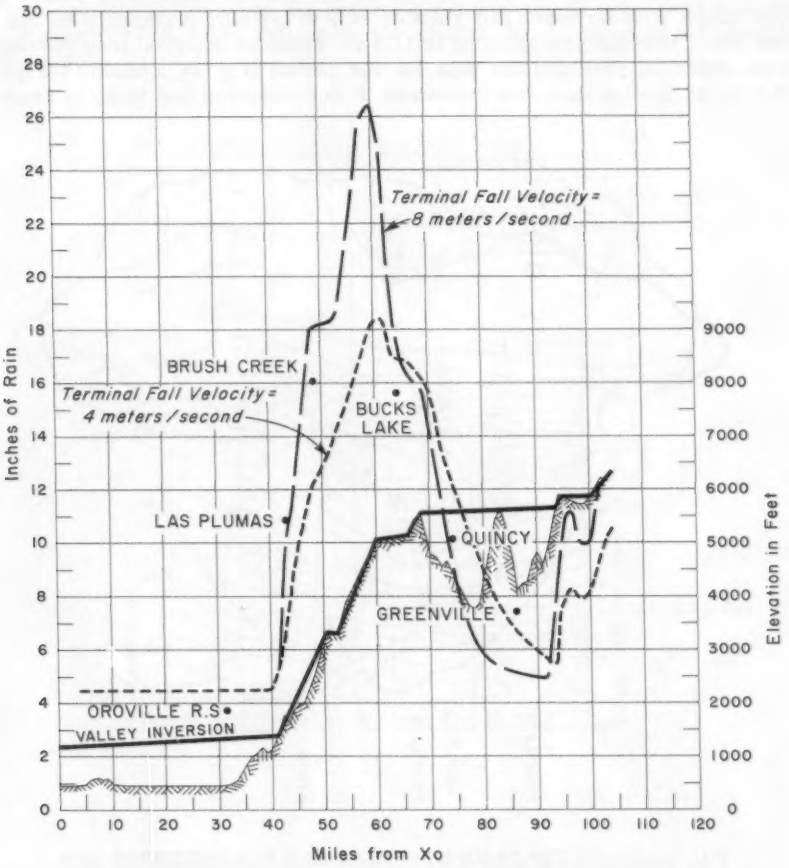
Terminal Fall Velocity = 8 meters/second
Terminal Fall Velocity = 4 meters/second
BRUSH CREEK
BUCKS LAKE
LAS PLUMAS
QUINCY
GREENVILLE
OROVILLE R.S.
VALLEY INVERSION
Inches of Rain
Elevation in Feet
Miles from Xo
0 10 20 30 40 50 60 70 80 90 100 110 120
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30
0 1000 2000 3000 4000 5000 6000 7000 8000 9000

FIG. 6.—SIMPLIFIED TERRAIN PROFILE AND COMPUTED 72-HOUR RAINFALL AMOUNTS FOR DECEMBER 1955 STORM, PROFILE "B," FEATHER RIVER BASIN

Test of Model On December 1955 Storm.—Based on upper air data observations during the December 1955 storm, the parameters required for computing the orographic precipitation and frontal precipitation were obtanied. By machine computation, the accumulated 72-hr precipitation for the three assumed rain drop terminal fall velocities for the period ending 0700 Pacific Standard Time (PST) December 22, 1955, were computed for the three profile strips across the Feather River Basin. The total 72-hr precipitation amounts and

simplified terrain profile for Profile "B," for assumed rain drop terminal fall velocities of 4 and 8 m per sec are plotted on Fig. 6. For comparison, precipitation data obtained from six stations within the 16-mile wide strip for the same 72-hr period are plotted on the plate as individual points.

The corresponding computed isohyetal map for the same 72-hr period (for rain drops with terminal fall velocity of 6 m per sec) is depicted on Fig. 7. The basin average precipitation is 11.8 in. From an isohyetal map developed from observed precipitation data for the period (Fig. 8), a basin average of 10.1 in. of precipitation was measured. It is considered that there is reason-

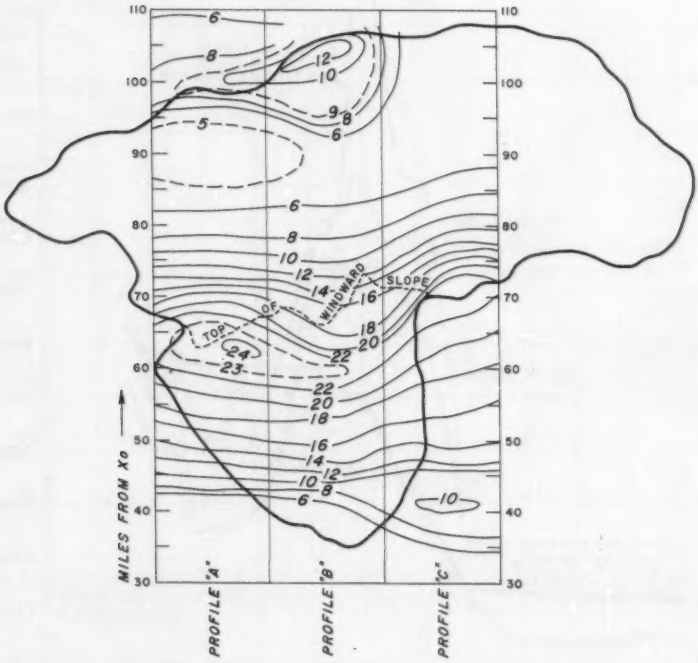

FIG. 7.—COMPUTED 72-HR ISOHYETAL MAP FOR DECEMBER 1955 STORM, FEATHER RIVER BASIN

able agreement between the computed and observed precipitation values, particularly since there is a paucity of observed data in the areas of maximum precipitation.

Estimated Probable Maximum Storm.—If the probable maximum storm is defined as the largest accumulated precipitation that can reasonably be expected to occur over the basin in a 72-hr period, it is necessary to combine the extreme meteorological parameter values in a reasonable and consistent manner. (The Knox report to the Department of Water Resources uses the term "maximum possible," whereas the term "probable maximum" is also widely used. It is considered that the two terms are synonymous.) This definitely does

not imply that all parameters are maximized simultaneously. By combining the parameters in a reasonable manner, an estimated maximum possible 72-hr precipitation total of 19.3 in. for the Feather River Basin was obtained. This total was based upon using assumed rain drops with terminal fall velocity of 8 m per sec. Total water on the windward slope is 34.1 in. and 15.8 in. on the leeward portion of the basin. By assuming a terminal fall velocity of 8 m per sec the amount of water falling on the critical frontal slope of the basin is maximized.

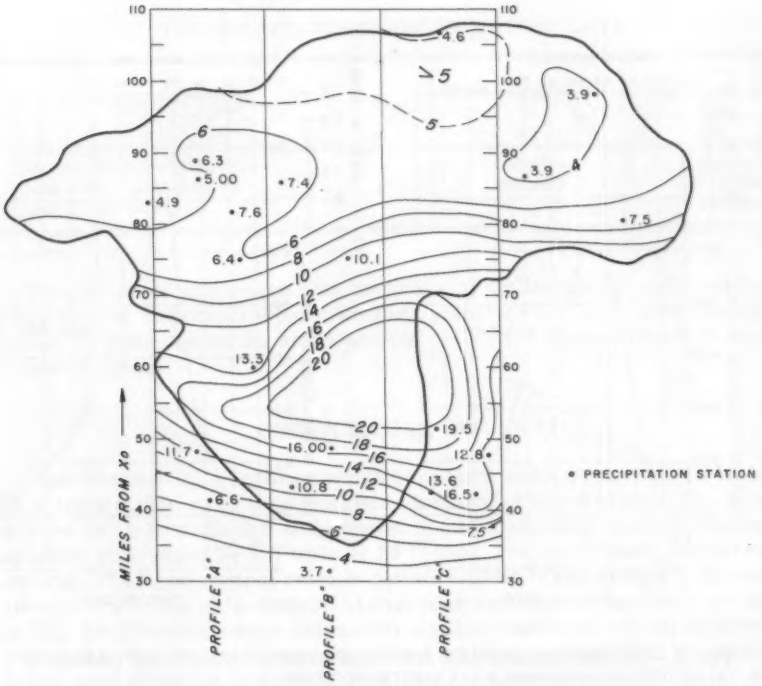

FIG. 8.—OBSERVED 72-HR ISOHYETAL MAP FOR DECEMBER 1955 STORM, FEATHER RIVER BASIN

Runoff from Probable Maximum Storm.—In order to compare the resulting runoff from the Department's studies with the Corps of Engineers' studies, all unit hydrographs, snowmelt contribution, and loss rates were used as developed in the Corps' hydrology study. However, the area and time distribution of the precipitation as determined by the Knox study was utilized. The resulting hydrograph for Oroville Dam Site has a peak flow of 615,000 cfs and a five-day volume of 2,932,000 acre-ft. The hydrograph and water excess (rain plus snowmelt minus losses) for the storm period are depicted on Fig. 9.

Probable Maximum Runoff Based on U.S. Weather Bureau Storm Studies.—Based on maximum probable storm studies by the Hydrometeorological Section

of the U.S. Weather Bureau published in 1943 and revised in 1957, the Department of Water Resources made a preliminary determination in 1957[5] of the resulting flood runoff. Using the 72-hr basin mean precipitation for the Feather River Basin of 21.1 in. as developed by the United States Weather Bureau Dept. of Commerce (USWB), and assuming conservative values of basin loss rates, snowmelt contribution, and so on, the computed runoff hydrograph resulted in a peak flow of 775,000 cfs and a five-day volume of 3,400,000 acre-ft.

The Sacramento District Corps of Engineers made a detailed analysis of the hydrology of the Feather River Basin in 1958. This analysis, also based on the precipitation data supplied bytthe USWB, resulted in a probable maximum flood

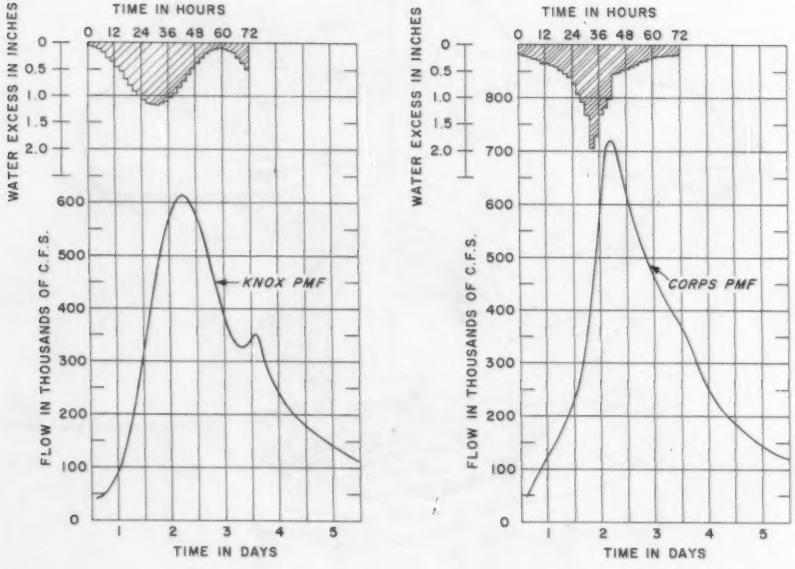

FIG. 9.—WATER EXCESS AND HYDROGRAPHS AT OROVILLE DAM SITE FOR PROBABLE MAXIMUM STORMS

hydrograph (Fig. 9) for the Oroville Dam Site with a peak flow of 720,000 cfs and total volume of 3,063,000 acre-ft. This design flood has been adopted by the Department of Water Resources and has been the basis for hydraulic considerations in the design of the spillway and the establishment of freeboard requirements for Oroville Dam.

Standard Project Flood.—The Sacramento District Corps of Engineers has developed "standard project floods" for the Feather River Basin and the Yuba and Bear Rivers. The following is a definition of standard project rain flood:

> "The standard project rain flood represents the runoff from a standard project rainstorm occurring on ground conditions that are reason-

[5] "Probable Maximum Flood at Oroville Dam Site," Office Report, State of California, Dept. of Water Resources, Sacramento, Calif., September, 1957.

able likely to exist during storms of such magnitude. The standard project rainstorm for a particular drainage area reflects the most severe flood-producing rainfall depth-area-duration relationship and isohyetal pattern of any storm that is considered reasonably characteristic of the hydrologic region in which the drainage basin is located, consideration being given to the runoff characteristics and effect of water regulation structures in the basin."

The computed peak flows and five-day volumes for the standard project floods for the three basins are shown in Table 1.

TABLE 1.—STANDARD PROJECT FLOOD DATA

Basin (1)	Peak Flow, in cfs (2)	Volume, in acre-feet (3)
Feather River at Oroville	440,000	1,944,000
Yuba River at Mouth	280,000	1,030,000
Bear River at Mouth	91,000	284,000

The Corps of Engineers and the State of California have agreed that the levee system now constructed on the flood plain together with upstream storage should be coordinated so as to provide protection against floods of standard project magnitude.

OROVILLE DAM SPILLWAY

The Oroville Dam Spillway and flood control outlet structure will be located in a topographic saddle on the right abutment of the dam site. The structure will be 700 ft in width and will consist of 380 ft of radial gate-controlled ogee spillway and seven 26.7 ft wide by 26 ft high, low level, flood control release outlets. The flood control release outlets will be in the center of the spillway structure with the gate-controlled overpour spillways on each side as depicted in Fig. 10. The four radial gates in the spillway sections will be 95 ft by 34 ft. The gates will be automatically raised when the water surface exceeds the gross pool elevation of 900 ft and will be fully opened when the water surface reaches 905 ft. The crest of the overflow spillway will be at elevation 868.0 ft United States Geological Survey, (USGS) Datum and the flood control outlets will be seated at elevation 821.5 ft, USGS Datum. The spillway chute will be concrete lined for a distance of approximately 400 ft downstream from the gate structure. From that point to the end of the spillway chute at the river, approximately 4,000 ft, the spillway channel will be excavated to bed rock and will be unlined. There is a fall of approximately 600 ft in this distance. Rock protection dikes will be required in part of the spillway chute.

Model studies of the spillway were conducted in 1955 by the Department of Water Resources to check design features and hydraulic characteristics of the spillway structure. Fig. 11 is a photograph of the spillway model. Based on these studies, the seven flood control release outlets will pass 249,000 cfs, and the spillway will pass 260,000 cfs, a total of 509,000 cfs, when the reservoir water surface elevation is at 900 ft. With water surface elevation at 909 ft, the

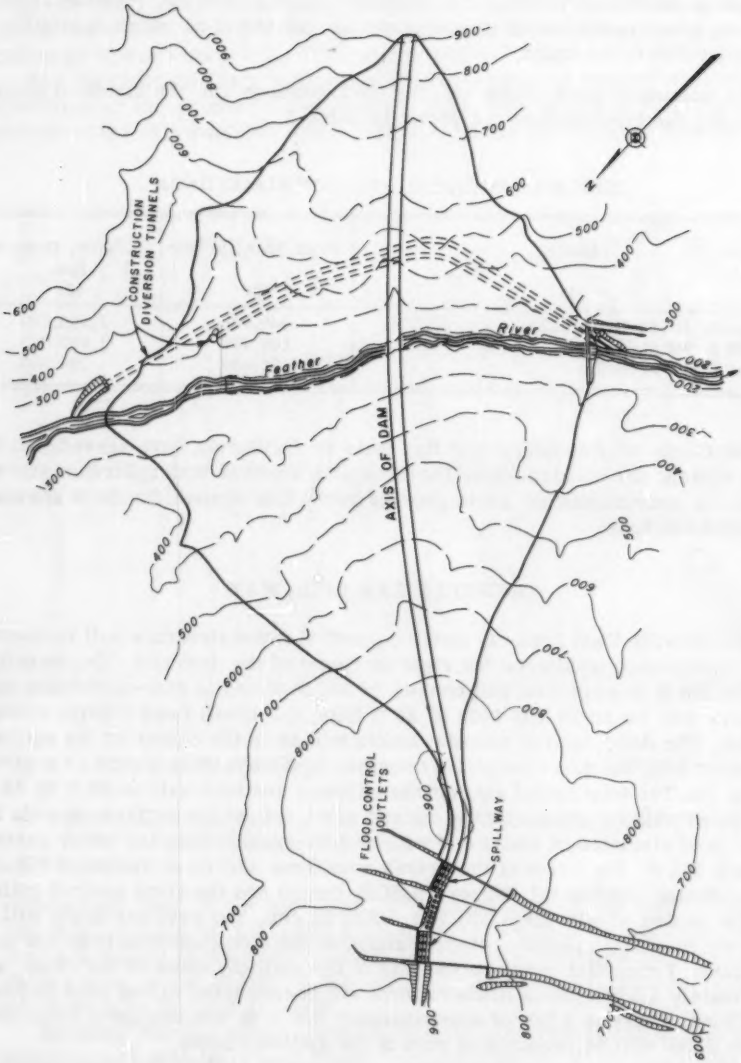

FIG. 10.—PROPOSED OROVILLE DAM AND SPILLWAY

combined discharge will be 650,000 cfs. Assuming that the maximum probable flood occurs with the reservoir water surface at gross pool elevation of 900 ft, the water surface would rise to an elevation of 909 ft. Crest of the dam will be 915 ft. Additional model studies of the spillway design and performance are scheduled by the Department of Water Resources for 1960-61.

FLOW DIVERSION DURING CONSTRUCTION

River diversion during construction will be accomplished by two 38-ft diameter tunnels, located in the left abutment. Floods will be permitted to pass over the dam embankment at the end of the first year of embankment construction if the diversion tunnels are overtaxed. The construction schedule requires that the embankment be placed above elevation 563 ft at the end of the

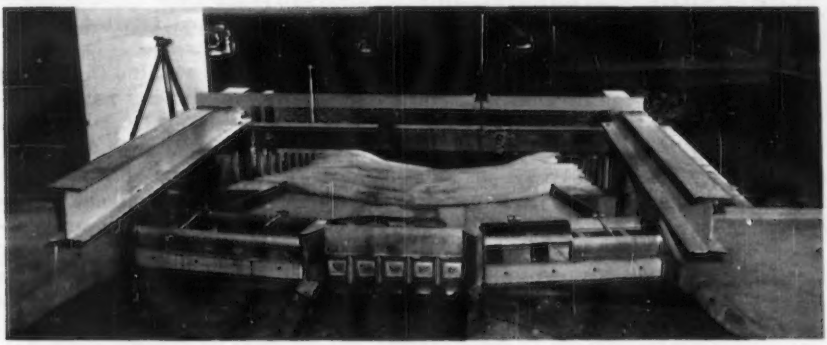

FIG. 11.—MODEL OF OROVILLE DAM SPILLWAY

second year of embankment construction in order that the standard project flood could be passed by the diversion tunnels without over-topping the embankment.

FLOOD CONTROL OPERATION CRITERIA

To meet the flood control objectives for the Feather River system, flood control releases from Oroville Reservoir must be limited to a maximum release of 150,000 cfs with the occurrence of a flood comparable to the standard project flood. To meet this requirement, the Corps of Engineers has developed a preliminary flood control reservation diagram that requires that a minimum flood control reservation of 375,000 acre-ft must be available between October 1 and April 1 of each year. Based on antecedent rainfall in the basin, a maximum reservation of 750,000 acre-ft may be required between October 15 and April 1. The exact amount of storage between these limits is based on the preceding 60-day basin-mean precipitation. The flood control reservation space can be filled between May 5 and June 15 each year. The exact date of permissible filling is also based on an antecedent rainfall parameter.

It is estimated that with the flood control storage reservation of 750,000 acre-ft, the average annual flood control benefit accruing to the Oroville Dam Project will be $3,640,000. Preliminary estimates of the federal government participation in construction costs of Oroville Dam and Reservoir, based on this flood control benefit, will be approximately 26% of the construction costs, exclusive of the cost of power facilities. This contribution, based on present (1959) cost estimates, will be approximately $77,000,000.

OPERATION OF OROVILLE DAM

The physical operation of Oroville Dam and Reservoir during flood periods will require special attention because of the very rapid runoff from the frontal slope of the Feather River Basin. Under existing conditions, the peak flow occurs at the dam site only 4 hr to 6 hr after the heaviest burst of precipitation. With the dam completed, this time will be shortened because of the effect of the extensive reservoir area. This condition will require the rapid accumulation of precipitation data from the basin and accurate and detailed weather forecasts.

The USWB in Sacramento is located in adjoining offices with the Operations Branch of the Department of Water Resources. The two agencies have established a Federal-State River Forecast Center which facilitates the rapid assimilation of meteorologic and hydrologic data and preparation of forecasts. In February, 1960, the USWB put in operation in Sacramento a WSR-57 radar. This is the newest type of radar developed for weather detection and is presently (1959) the only one in the western United States. It is a 10-cm set operating with a peak power of 500,000 watts. This set gives excellent coverage of a large part of the Central Valley watershed. As experience is gained with the information developed from the radar installation. it is anticipated that more accurate runoff forecasting will be possible.

In addition to an extensive radio reporting stream gage network operated by the Department of Water Resources in the Central Valley, an experimental radio reporting rain gage was installed at Brush Creek in 1959. This gage has proved to be very reliable. (The precipitation data recorded on Fig. 3 was obtained from this radio reporting rain gage.)

It is anticipated that the combination of the radar and radio reporting river and rain gages will provide the hydrologists and meteorologists with the information to operate efficiently Oroville Dam and Reservoir and to integrate its operation with the operation of the reservoir and levee systems in the Central Valley.

ACKNOWLEDGMENTS

The contribution to the field of hydrometeorology by Mr. Joseph B. Knox and his work with the Department of Water Resources are gratefully acknowledged. The cooperation of the Sacramento District, Corps of Engineers, is also appreciated. The figures presented herein were drawn by Mr. Kay Shibata, Delineator, Department of Water Resources, State of California.

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Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

SACRAMENTO RIVER DEEP WATER SHIP CHANNEL: FUNCTIONAL PLANNING[a]

By Amalio Gomez,[1] F. ASCE

SYNOPSIS

This paper describes the functional planning studies made for the Sacramento River Deep Water Ship Channel, now under construction by the Corps of Engineers. The paper includes a description of the project, functional requirements, hydraulic studies, estimates of benefits, and comparison of benefits and costs.

INTRODUCTION

The oldest record of navigation on the Sacramento River is that left by the Arguello expedition. A party of some 60 men left San Francisco on October 18, 1821, proceeding by boat near to Colusa, then by land to the vicinity of Cottonwood Creek, crossing the Coast Range and returning to San Francisco on November 15. The expedition was in search of new mission sites and "foreigners" who were said to have settled in the area, but it found neither. The first improvement of the Sacramento River for navigation came 54 yrs later in 1875, but it consisted mainly of the removal of snags. Since that time the river has been progressively improved. Existing controlling depths are 10 ft to Sacramento, 6 ft to Colusa, 5 ft to Chico Landing, and such depths as are practical from Chico Landing to Red Bluff.

As early as the beginning of this century, serious thought was being given to the practicability of providing deep water navigation up to the city of Sacramento. The record shows that in 1916, the State Department of Engineering ap-

Note.—Discussion open until April 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. WW 4, November, 1960.

[a] Presented at the June, 1960 ASCE Convention in Reno, Nev.

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proved the expenditure of $7,000 for a survey and investigation to determine the feasibility of constructing a ship canal between the city of Sacramento and deep water. The survey was begun in 1916, under the direction of Major Paul M. Norbee, then Assistant State Engineer. The investigation, suspended during the war years, was completed in 1922. Several routes were investigated by Mr. Norbee, including the one along which the channel is being built. The route selected as the most practical at that time by Mr. Norbee followed the San Joaquin River upstream to near the mouth of the Mokelumne River, along an excavated canal in a northeasterly direction upstream between the Mokelumne River and the Sacramento River, to near the junction of the Cosumnes and Mokelumne Rivers, and thence northerly across low lying land to Sacramento.

A report on the economic aspects of a Sacramento deep water ship channel was prepared in 1925, by C. E. Grunsky, Consulting Engineer for the Sacramento Deep Water Commission. Mr. Grunsky adopted the route suggested by Mr. Norbee, and concluded that whereas the project could not be justified from a restricted financial standpoint, the region in fair proximity to the canal and port would be warranted in carrying out the project.

The ship channel project was authorized on the basis of the next detailed report on the subject, prepared in 1945, by the District Engineer, Sacramento District, Corps of Engineers. This report, as well as the reports of the Division Engineer, Board of Engineers for Rivers and Harbors, and the Chief of Engineers, is printed as Senate Document No. 142, 79th Congress, 2nd session. It recommended construction of a 30-ft channel up the Sacramento River and Cache Slough and then overland, west of the river to a harbor at Washington Lake, near Sacramento. Congress authorized construction of the project in the River and Harbors Act approved on July 24, 1946. Construction was initiated in 1949, but was suspended during the Korean War. It was renewed in 1956, and will be completed in about 1963, thus culminating a 47-yr effort since Mr. Norbee began his investigation in 1916. Fig. 1 shows a diagrammatic layout of the project area.

DESCRIPTION OF THE PROJECT

The project will provide a ship channel 30 ft deep at mean lower low tide and 200 ft to 300 ft wide at the bottom, from deep water in Suisun Bay (an arm of San Francisco Bay) to Washington Lake near Sacramento; a harbor and turning basin in Washington Lake; and a connecting canal 13 ft deep at lower low tide and 120 ft wide at the bottom, with an 86-ft x 640-ft navigation lock between the harbor and the Sacramento River. The ship channel will be formed by widening and deepening the Sacramento River and Cache Slough for the first 18 miles above Suisun Bay and by excavating a new channel just inside the easterly boundary of Yolo Bypass (a leveed floodway for excess Sacramento River water) for the remaining 25 miles to Washington Lake, (a natural body of water unconnected with the river). The harbor and turning basin, formed by the deepening and enlarging of the existing Washington Lake, will be 30 ft deep at mean lower low tide, with an initial bottom area of 70 acres and provisions for future enlargement. The 1-1/2-mile long barge canal and navigation lock will permit the transit of shallow draft vessels between the deep-draft harbor and the river, regardless of the difference in water surface elevations, except during extreme flood flows when the river becomes unnavigable; at

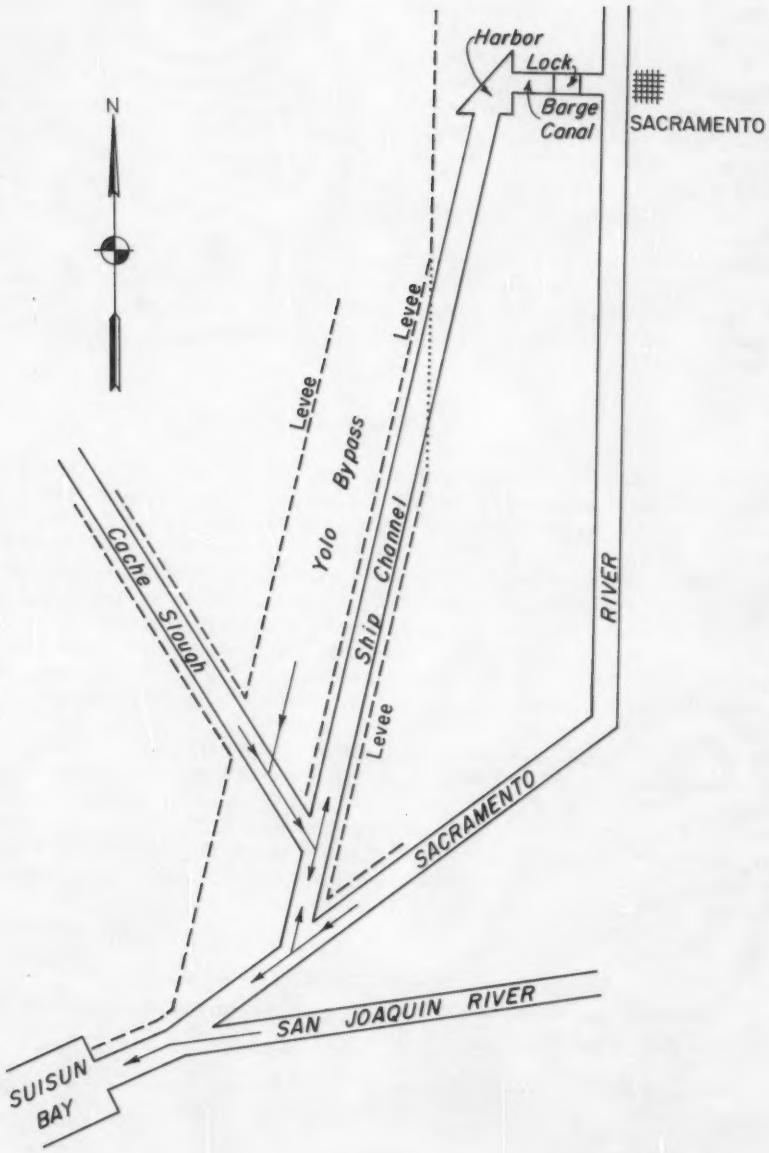


FIG. 1.—DIAGRAMMATIC LAYOUT

this time, the canal will be closed to traffic. A single leaf combination highway and railroad bascule bridge will cross the canal at the harbor end of the navigation lock.

Local interests will construct, operate, and maintain an adequate public terminal with necessary utilities and rail and highway connections, at the harbor open to all on equal terms. They will also furnish without cost to the United States all necessary lands, easements, rights-of-way, and spoil disposal area for initial work and subsequent maintenance, and make all utility changes. Local interests are also required to hold and save the United States free from all claims for damages arising from the construction, operation, and maintenance of the project.

The State of California, by Senate Bill No. 730, Chapter 1152, 1947 Session, approved an appropriation of $750,000 for defraying a portion of the cost of real estate acquisition. The Sacramento-Yolo Port District has issued bonds in the amount of $3,750,000 to provide for the other requirements of local cooperation.

FUNCTIONAL REQUIREMENTS

Route.—Preliminary studies indicated that three routes offered the most possibilities, and these were investigated in detail. These routes included the one east of the river, proposed by Mr. Norbee and Mr. Grunsky in the twenties, a route following the river, and the selected route west of the river. The route east of the river was abandoned because of excessive excavation, expensive rights-of-way, and generally undesirable location. The river route was the least desirable of the three because it would have been the most costly, the most tortuous, and about 15 miles longer than the adopted route. The selected route west of the river had the advantages of least cost, shortest distance, and most desirable location, particularly with respect to harbor and terminal facilities.

Channel.—The depth adopted for the design of the Sacramento River Deep Water Ship Channel is 30 ft below mean lower low water, that is approximately elevation +1 on Corps of Engineers datum and -2 on mean sea level datum. Mean lower low water is at approximately the same elevation throughout the length of the channel. Since the channel is in tidal water, the actual depth available will be in excess of 30 ft practically all the time. Table 1 illustrates the percentage of time that a given depth is available:

TABLE 1.—PERCENTAGE OF TIME A GIVEN DEPTH IS AVAILABLE

Water surface elevation, in feet		Water depth, in feet	% of time depth is exceeded
C of E datum	M S L datum		
+1	-2	30	100
+2	-1	31	95
+3	0	32	85
+4	+1	33	50
+5	+2	34	30

The vessels calling at Sacramento are expected to be about the same as those calling at Stockton, that also has a 30-ft channel. For the year 1955, the vessels calling at Stockton were approximately as they are given in Table 2. When these vessels are fully loaded and underway, they require about 4-ft or 4.5 ft additional depth under the keel. For instance, a C-3 vessel fully loaded and underway requires a total depth of 28.5 + 4.5 = 33 ft that, for the Sacramento River Deep Water Ship Channel, is available 50% of the time, Such a vessel, if fully loaded, would occasionally have to wait 2 hrs. or 3 hrs for medium or high tide. The selected channel width is 300 ft from mile 0.0 at Suisun Bay to mile 18.5 at the mouth of Cache Slough. From Cache Slough to the harbor at mile 42.8, the channel width is 200 ft on tangents and 300 ft on curves. These channel widths were based on experience at the Port of Stockton and elsewhere. Side slopes fo the cut vary from 1 on 3 to 1 on 4. Land acquisition and other controlling elements of the project are such as to permit future deepening of the channel to 35 ft and widening to 300 ft on tangents and 400 ft on curves.

Turning basins.—From Suisan Bay to Cache Slough, there are several locations at which a deep-draft vessel could be turned in case of an emergency.

TABLE 2.—VESSELS CALLING AT STOCKTON

Type of vessel	No. of vessels, in %	Draft When Fully Loaded, in feet
C-4	15	32.8
C-3	37	28.5
C-2	40	25.7
C-1	3	27.5
Other	5	-
Total	100	-

The uppermost of such locations is at about mile 18.6 in Cache Slough, near the mouth of Miner Slough. In view of this fact, no specific turning basins are being constructed. However, the levees in the vicinity of mile 32 have been set back far enough to permit construction of a future turning basin at that location, in the event one is found necessary.

Harbor.—In selecting the size of the harbor, consideration was given to the dimensions of the prospective vessels; terminal facilities to be provided by local interests; area of harbor and turning basin of the Port of Stockton; future expansion plans; and desires of local interests. The selected harbor is triangular in shape and has an area of about 70 acres. It can be expanded in the future to at least twice its initial area. The depth is 30 ft below mean lower low water.

Barge canal.—The barge canal is 13 ft deep at mean lower low water, and 120 ft wide at the bottom, with provisions for future widening to 160 ft. Its purpose is to permit shallow-draft traffic to move from the harbor to the Sacramento River, and vice versa. The canal dimensions were selected after consideration of the size of existing barges and tugs operating in the San Francisco Bay area and on the Sacramento River; the size of the self-propelled barges that are proposed for construction by San Francisco Bay navigation interests; and the possibility of future enlargement of the canal.

Lock.—Water levels in the harbor will vary from elevation zero to el + 18, Corps of Engineers datum. Water levels in the Sacramento River at its junction, with the ship channel through the barge canal, vary from el + 2 at low water to el + 33 during flood periods. A lock is, therefore, necessary in the barge canal. Although the lock gates will occasionally be subject to reversal in water pressure, about 95% of the time the water level in the river will be higher than in the ship channel by a difference varying up to about 21 ft. Considerable study was made to determine lock dimensions. Preliminary studies made in 1944 indicated that such a lock should be approximately 60 ft x 425 ft x 11 ft. More refined studies were made in 1948, and all prospective users of the lock were contacted on an individual basis. As a result of these studies and contacts, and taking into account the experience gained from operating other locks throughout the country, the lock dimensions were changed to 86 ft x 640 ft x 13 ft. Such dimensions will permit the operation of two barges in tandem with a tow boat, or of a single barge with a tow boat abeam.

Enlargement of Yolo Bypass Channels.—Construction of the ship channel, with its high guard levees, up the favorable alignment within Yolo Bypass, encroaches on the waterway area of that flood-control channel and requires some compensatory enlargement of the remaining bypass. This compensation was accomplished by widening the deep channel of Cache Slough, clearing and deepening two of the interior slough through the narrow lower end of Yolo Bypass, and dredging a tow drain along the bypass side of the ship channel guard levee for the remaining distance upstream. Hydraulic computations indicated that these various works in combination would largely compensate for the encroachment of the ship channel into the floodway and prevent any significant rise in its project flood plane. This enlargement program was undertaken in advance of the main ship channel construction in order to avoid risk during the construction period.

Associated problems.—In addition to the primary problems of selection of route, determination of the dimensions of the various units of the project, and providing compensatory enlargement of Yolo Bypass, there were numerous subordinate problems to be solved. The ship canal cuts into the lower end of Yolo Bypass at which there is a maze of tidal sloughs separating intensively cultivated islands that are protected against high tide and moderate flood flows by low levees. Ship channel construction will block certain of these sloughs and concentrate flow in other sloughs. Hydraulic analysis of the rapidly changing tidal and flood flows in the existing and future networks of interconnecting channels was extremely difficult. After long study it was found necessary to modify some of the slough junctions and to revet the banks at other junctions. Another problem was the possible effect of the new tidal prism in the ship channel on tidal conditions throughout the rest of the delta area. Much of this delta is near the upper limit of saline intrusion from San Francisco Bay, and changes in tidal conditions may adversely affect the quality of irrigation water used by the delta tracts. Theoretical studies of saline intrusion indicated that, although the ship channel would increase the tidal prism by only about 6%, compensatory work might be necessary to prevent adverse salinity effects downstream. Accordingly, the sum of $2,000,000 was included in the original project cost estimate for this purpose. The problem is confused by numerous man-made changes that have occurred in the delta and plans for still larger changes in the future. Decision as to whether compensatory works will be necessary has been deferred until more concrete evidence becomes available.

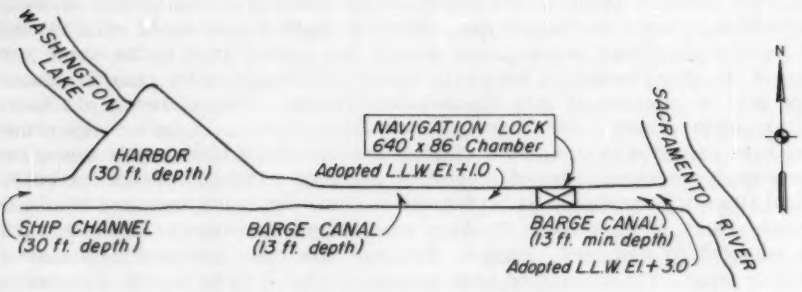
Another problem involves the effect of the increased tidal prism on sedimentation patterns in downstream channels, particularly in the main channel of Sacramento River below Cache Slough that is traversed by the lower portion of the ship channel. This tidal channel has been shoaling, and this condition may be affected by ship channel construction. Consideration of this indeterminate problem has been deferred until some maintenance experience has been gained with completed channel conditions. A final problem involves the possible accumulation of industrial and ship wastes in the harbor water. Tidal flows will probably be inadequate to flush out foul water, and it may be necessary to provide more flushing water from Sacramento River than will be supplied by ordinary lockage. The lock has been designed with such a need in mind. The amount of flushing water used will be kept to a minimum because of the high value of fresh water in this region. No special provisions were made in the interest of fish, but passage of fish between the river and the harbor will be possible through the lock.

BASIC HYDRAULIC STUDIES

The Sacramento River at Sacramento is subject to tidal fluctuations varying from approximately 3 ft at low water to practically nothing when the flow in the river exceeds 25,000 cfs. The estimated tidal range in the deep water channel at the harbor varies from approximately 6 ft during low water periods to less than 1 ft during extreme flood conditions in lower Sacramento River. Tidal range data are particularly important in establishing the bottom elevations of the channel, the harbor, and the barge canal. Tidal range data are also important in determining the differential head at the lock between the river and the harbor.

Figs. 2 and 3 contain a series of charts summarizing the hydraulic and tidal range studies that have been made. Fig. 2(b) is a simple river rating curve. Fig. 2(c) shows a comparison of the typical tidal cycle at both ends of the lock. The cycle at the river end is an observed cycle. The cycle at the harbor end is derived from the observed tidal cycle at Rio Vista by a theoretical, reach-by-reach computation up the Sacramento River to Junction Point, up the improved Cache Slough to the foot of the new channel, up the new channel to the harbor, across the harbor to the barge canal, and up the barge canal to the lock. Computations were based on the Pettis formulas and have subsequently been approximately checked on an electrical analog model built by the University of California. Both these methods indicate that there will be about 1 ft more tidal range in the harbor at the head of the ship channel than in the open river at the foot of the ship channel. High tide will be about 1/2 ft higher and low tide 1/2 ft lower, as has been observed in other dead-end channels of large cross section. A complete series of these tidal cycles was developed to cover the full range of concurrent stanges in river and harbor. Fig. 2(d) represents the high and low values from such a series of cycles. Fig. 2(e) is a duration curve of differential heads between the river end and harbor end of the lock developed from hourly values taken from the same series of cycles.

Fig. 3(a) contains flow-duration data for the Sacramento River developed from historical discharge records adjusted to present conditions of upstream storage. The high tide curve was derived from a similar adjusted stage record counting only the highest stage on each day. Similarly, the low tide curve



(a) LAYOUT OF HARBOR, LOCK, AND RIVER

SCALE IN FEET
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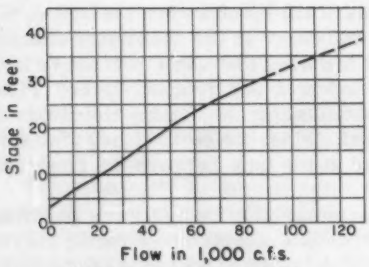
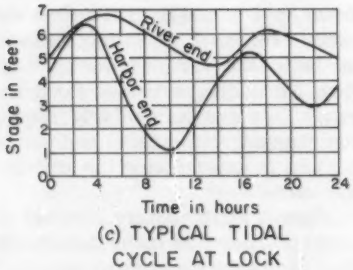
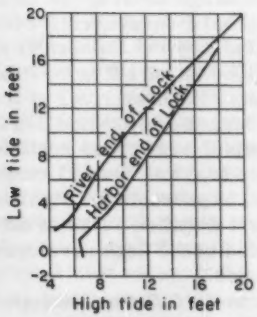


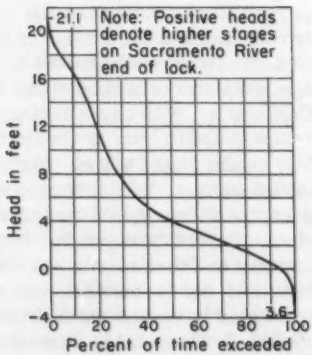
(b) RATING CURVE FOR SACRAMENTO RIVER



(c) TYPICAL TIDAL CYCLE AT LOCK

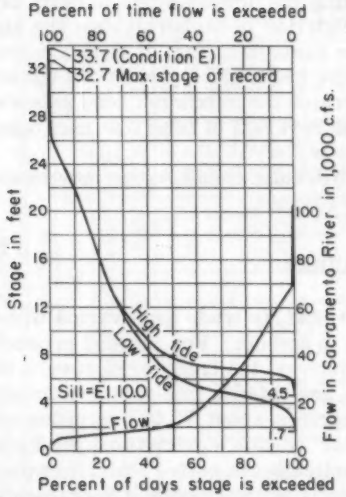


(d) AVERAGE TIDAL RANGE AT LOCK

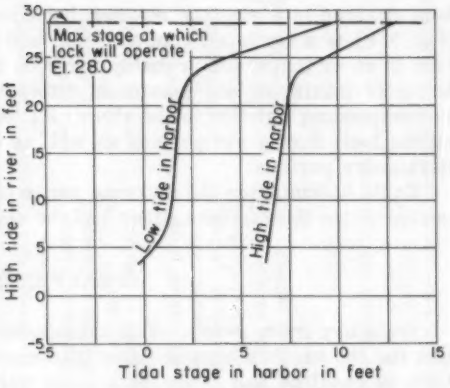


(e) DIFFERENTIAL HEAD DURATION AT LOCK

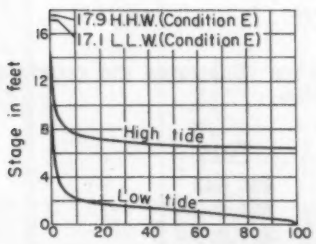
FIG. 2.—HYDRAULIC DESIGN DATA



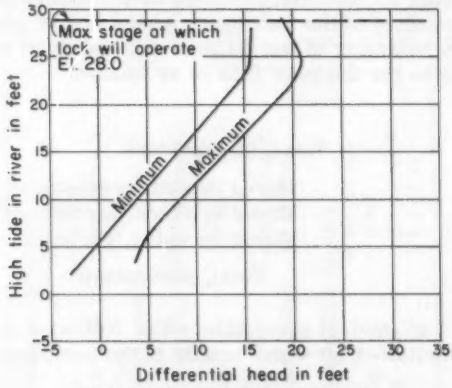
(a) STAGE AND FLOW DATA AT RIVER END OF LOCK


Max. stage at which lock will operate El. 28.0
High tide in river in feet
Low tide in harbor
High tide in harbor
Tidal stage in harbor in feet

(c) RANGE OF STAGES IN HARBOR



(b) STAGE DATA AT HARBOR END OF LOCK



(d) RANGE OF DIFFERENTIAL HEAD ACROSS LOCK

FIG. 3.—HYDRAULIC DESIGN DATA

was derived from the same adjusted stage record counting only the lowest stage on each day. The high tide curve of Fig. 3(b) was derived from stage record at Rio Vista, adjusted for the computed difference between Rio Vista and the harbor, and counting only the highest stage on each day. The low tide curve was derived in a similar manner but counting the lowest stage in each day. Fig. 3(c) is a correlation of low tide and high tide in harbor against the high tide in river that would probably occur on the same day. Fig. 3(d) is a correlation of minimum and maximum differential heads during each day against corresponding high tide in the river. It illustrates the maximum head expected on the lock during wet periods as well as the reversal of head that may occur during dry periods.

Table 3 illustrates the extreme range of hydraulic condition that may be expected in the Sacramento River and the ship channel.

TRAFFIC STUDIES

Tributary trade areas.—The Sacramento tributary trade area varies in extent for the various commodities that move in and out. For inbound products such as gasoline and other petroleum derivatives, the trade area covers the entire interior portion of northern California and extends into southern Oregon and western Nevada. This trade area comprises about 75,000 sq miles and contains a population of about 1,000,000. For certain agricultural products, such as almonds, the trade area includes practically the entire State, inasmuch as most of the California production of almonds is processed in Sacramento. A summary of the inbound and outbound commerce for the Sacramento trade area for the year 1952 is as follows:

	Inbound	Outbound
Total, in 1,000 tons	5,600	8,200
Moved by rail, percentage	14	52
Moved by truck, percentage	57	45
Moved by water, percentage	29	3
Total, percentage	100	100

Historical navigation.—The following tabulation illustrates the growth of shallow-draft water traffic in the Sacramento River:

Year	Tons
1930	853,648
1935	632,599
1940	817,235
1945	956,489
1950	1,590,883
1955	2,228,431
1956	1,886,540
1957	2,703,529
1958	2,259,495

TABLE 3.—STAGE DATA SACRAMENTO RIVER SHIP CHANNEL

Location	Condition									
	A		B		C		D		E	
	Water surface elevation[a]									
	LLW	HHW	LLW	HHW	LLW	HHW	LLW	HHW	LLW	HHW
Sacramento River at Barge Canal	1.7	4.5	4.6	6.7	29.0	29.0	29.0	29.0	33.7	33.7
Barge Canal at Sill on River end of lock	1.7	4.5	4.6	6.7	29.0	29.0	29.0	29.0	33.7	33.7
Barge Canal at Sill on Harbor end of lock	-0.5	6.7	1.0	6.4	3.7	9.2	5.3[b]	10.7[b]	17.1	17.9
Turning Basin	-0.5	6.7	1.0	6.4	3.7	9.2	5.3[b]	10.7[b]	17.1	17.9
Ship Channel at Mile 40	-0.4	6.7	1.1	6.4	3.8	9.1	5.3[b]	10.6[b]	17.1	17.9
Bypass at Mile 40	dry	dry	dry	dry	dry	dry	23.0[b]	23.0[b]	27.9[b]	27.9[b]
Confluence of Channel and Cache Sl. at Mile 18	0.4	6.0	1.7	5.9	4.4	8.6	5.9[b]	10.1[b]	17.2	17.8
Confluence of Cache Sl. and River at Mile 14	0.4	6.0	1.7	5.9	4.4	8.6	4.5	8.7	14.1	15.5
Sacramento River at Rio Vista	0.4	6.0	1.7	5.9	4.3	8.5	4.3	8.5	11.8	13.6

A-Minimum Flow Conditions (Shasta passing inflow)
- Flow in River at Sacramento = 0 cfs
- Flow in Yolo Bypass at Cache Sl. = 0 cfs
- Minimum tide range at Rio Vista = 6.0 to 0.4 ft
- Minimum tide range at Sacramento = 4.5 to 1.7 ft
- Probable frequency of exceed. (minimum) = 1 in 100 yr

B-Normal Low Flow Conditions (Normal operation of Shasta)
- Flow in River at Sacramento = 6,100 cfs
- Flow in Yolo Bypass at Cache Sl. = 0 cfs
- Average tide range at Rio Vista = 5.9 to 1.7 ft
- Normal tide range at Sacramento = 6.7 to 4.6 ft
- Probable frequency of exceed. (minimum) = 1 in 1 yr

C-Normal Flood Conditions (No flow in Yolo Bypass)
- Flow in River at Sacramento = 85,000 cfs
- Flow in Yolo Bypass at Cache Sl. = 0 cfs
- Average tide range at Rio Vista = 8.5 to 4.3 ft
- Probable frequency of exceed. = 1 in 2 yr

D-Normal Flood Conditions (No flow over Sacramento Weir)
- Flow in River at Sacramento = 85,000 cfs
- Flow in Yolo Bypass at Lisbon = 150,000 cfs
- Flow in Yolo Bypass at Cache Sl. = 155,000 cfs
- Average tide range at Rio Vista = 8.5 to 4.3 ft
- Probable frequency of exceed. = 1 in 2 + yr

E-Project Flood Conditions (Flow in both bypasses)
- Flow in River at Sacramento = 95,000 cfs
- Flow in Yolo Bypass at Lisbon = 499,000 cfs
- Flow in Yolo Bypass at Cache Sl. = 600,000 cfs
- Max. probable tide range, Rio Vista = 13.6 to 11.8 ft
- Probable frequency exceed. = More than 200 yr

[a] All elevations or stages are referred to C. of E. Datum = 3.0 ft below Mean Sea Level

[b] Estimated

Future deep draft traffic.—Extensive studies were made of expected future deep draft traffic. These studies were brought up-to-date in 1955. Table 4 indicates a summary of the results:

TABLE 4.—EXPECTED FUTURE DEEP DRAFT TRAFFIC.

Commodity or commodity group	Average annual traffic in 1,000 tons		
	1960	1985	2010
Outbound			
Rice	215	250	278
Barley	108	125	143
Canned goods	140	185	240
Dry beans	25	27	31
Dry fruits	24	28	33
Millfeeds	8	10	14
Miscellaneous	42	55	70
New products (10%)	56	68	81
Subtotal	618	748	890
Inbound			
Lumber	6	9	12
Steel products	25	36	48
Molasses	13	13	15
Fertilizer	10	10	12
Pineapple	15	15	17
Miscellaneous	14	16	23
New products (10%)	9	10	13
Subtotal	92	109	140
Grand total	710	857	1,030

Future shallow draft traffic.—Historically, shallow-draft traffic has consisted primarily of inbound gasoline and other petroleum products. This traffic is expected to continue moving by shallow draft because of the lack of sufficient terminal storage capacity, now or in the foreseeable future, to permit the use of tankers to transport the gasoline and petroleum products. Whereas this appraisal may not prove to be fully correct over the long range, it is the safest approach, in that it precludes overestimating the navigation benefits. In view of the shorter and better route provided by the deep water channel, it is expected that the bulk of the shallow-draft traffic to Sacramento and points north will use such channel. Projections of future petroleum use were related to population growth. On the basis of population trends, estimated future average annual tonnage over the 50-year period, 1960-2010, have been computed and the results are shown in Table 5.

NAVIGATION BENEFITS

Basic concept.—Navigation benefits are considered to represent the difference in the cost of transporting a given commodity by water as against the cost of transporting the same commodity by the cheapest alternative means, such as by truck or by rail, or by an alternative water route. The computation of

navigation benefits, therefore, resolves itself into a study of the cost of alternative means of transportation.

Basic data.—Studies indicate that the average boat calling at Sacramento will lift about 3,500 tons and that it will take about 20 hr of actual travel time for a round trip from San Francisco to Sacramento. For ships calling at both Stockton and Sacramento on the same trip, the difference in time between making one trip, calling at both ports, and two independent trips is 6-2/3 hr. It is estimated that approximately 160 vessels per year will call at both ports on the same trip. Vessel operating costs, under 1959 prices, for the type of vessels expected to use the deep water channel were estimated at $135 per hr, including all fixed and operating charges, pilot costs, and profit. The equivalent cost per ton of deep water cargo is 20 x 135/3500 = $0.77. The cost of moving the same commodities by the cheapest alternative means varies from $2.40 per ton of barley to $4.55 per ton of nuts, with a weighted average of $3.17 per ton. The corresponding navigation benefit is $2.40 per ton. For shallow-draft cargo, the benefits involved are the difference in cost of moving the cargo via the Sacramento River as against the cost of moving the same cargo via the deep water channel, at which the distance is shorter and the navigation conditions

TABLE 5.—ESTIMATED FUTURE AVERAGE ANNUAL TONNAGE.

Commodity or commodity group	Average annual traffic in 1,000 tons		
	1960	1985	2010
Gasoline	1,650	2,600	3,300
Other petroleum products	700	1,200	1,600
General cargo	150	200	250
Total	2,500	4,000	5,150

more favorable. Such a difference in cost has been estimated to vary from $0.123 per ton of gasoline to $0.144 per ton of general cargo, with a weighted average of $0.125 per ton.

Navigation benefits.—Average annual navigation benefits, based on July, 1959 price levels, are summarized below. Unit savings are weighted means.

Deep draft	
857,000 tons at $2.40	= $2,060,000
Shallow draft	
4,000,000 tons at $0.125	$ 500,000
Total	$2,560,000

COLLATERAL BENEFITS

Construction of the ship channel with its high and wide guard levees within Yolo Bypass will incidentally provide a substitute for a long reach of the eastern levee of that bypass. The latter levee does not meet project standards for the bypass levees, and flattening of its slopes to meet the standards is part of another authorized project. Consequently, the slopes of a portion of the

levee will not have to be flattened to the required standards. This represents an annual saving to the flood-control project of $54,000. Also the high ship channel levee will run in front of certain tracts now in Yolo Bypass and project them from flooding. The resultant increased degree of protection for these tracts is estimated at $133,000 per year. These collateral flood-control benefits total $187,000 per year.

The project document provides that, if construction of the ship channel aggravates salinity conditions in the delta because of the increase in the tidal prism, compensatory measures should be undertaken. It is by no means certain that such will be the case. However, the estimate of cost for the project carries an allowance of $2,000,000 for such possible eventuality. If it should become necessary to undertake such remedial work, the most practical and economical procedure would be the reclamation of one of the presently submerged delta tracts having a tidal prism volume approximately equal to that of the ship channel. Such reclamation would yield agricultural benefits in the order of $100,000 per year as a partial offset to the initial expenditure of $2,000,000 for salinity compensatory work.

The total collateral benefits amount to $287,000 per year on the average.

JUSTIFICATION

A summary of project costs, based on July, 1959 prices, is as follows:

Federal first cost	$43,400,000
Non-Federal first cost	3,515,000
Total first cost	$46,915,000
Federal annual cost	$ 2,046,000
Non-Federal annual cost	197,000
Total annual cost	$ 2,243,000

A summary of average annual project benefits, also based on July, 1959 price level, is as follows:

Navigation	$ 2,560,000
Collateral benefits	287,000
Total	$ 2,847,000

Comparing the average annual benefits of $2,847,000 with the average annual cost of $2,243,000 results in a benefit-cost ratio of 1.3.

CONCLUSIONS

Deep-water navigation to Sacramento has been seriously considered since the beginning of the century, and detailed studies were begun as early as 1916. The efforts by several agencies culminated in authorization for construction in 1946.

The functional design of the ship channel project is based on accommodating deep-water ships up to and including those of the C-3 class, together with the

existing shallow-draft vessels now using the Sacramento River. The comparison of benefits and costs indicated that construction to permit use by larger ships is not economical at this time, but provisions for possible future enlargement in depth and width were made in the functional design and in the acquisition of rights-of-way. In addition to the size of vessels, other important considerations involved in functional design were tidal fluctuations and the necessity for transfer of shallow-draft traffic to the Sacramento River at Sacramento. Such transfer required a barge canal and lock, that must provide for water level differences between harbor and river varying from about 2ft higher on the harbor side during minimum flow conditions, to about 25 ft higher on the river side during certain flood conditions.

The project will serve a tributary trade area of 75,000 sq miles with a population of 1,000,000 persons. It is estimated that by 1985, deep-draft navigation will amount to about 850,000 tons per year, and shallow-draft navigation to about 4,000,000 tons per year. The project cost is estimated at about $47,000,000 and the annual benefits are expected to exceed the annual costs by about 30%.

ACKNOWLEDGMENTS

Studies described herein were made by the United States Army Engineer District in Sacramento over a 15-year period. It is, therefore, difficult to give credit to the many fine military and civilian engineers that took part in the work. H. A. Morris is now the District Engineer. Frank Kochis, Chief of the Engineering Division throughout the entire period, provided general supervision. W. J. Parsons, Chief of the Hydrology Section, directly supervised the functional hydraulic studies, and A. G. Atkins, Chief of the Economics Section, Supervised the economic studies.

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

DESIGN CONSIDERATIONS FOR CALIFORNIA MARINAS

By James W. Dunham,[1] F. ASCE

SYNOPSIS

Considerations and criteria for the functional layout and structural design of marinas for small craft, as proposed by the California State Division of Small Craft Harbors, are presented. Examples of successful design practice are given, and recommendations are made for use of various types of material and design under different conditions of exposure.

INTRODUCTION

Until about 1950, the number of recreational boats in use throughout the United States was well below 3,000,000. Most of the relatively few publications that relate specifically to design of facilities for small craft were written prior to that time. According to statistics compiled by the Outboard Boating Club of America and the National Association of Engine and Boat Manufacturers, 6,954,000 recreational boats were in use in 1957, 7,330,000 in 1958, and 7,800,000 in 1959. Even the annual increment has been increasing by 100,000 boats a year. Facilities for launching, berthing, and otherwise accommodating these boats and catering to the needs of their owners have been constructed at a rate far too slow to meet the demands. Many of the existing facilities were planned and built by non-professional persons with little attention being paid to proper design, with the result that sub-standard marinas were often the rule rather than the exception. Only in the past few years has the civil engineering profession begun to awaken to the challenge presented by this problem. When the California State Division of Small Craft Harbors was created by legislative

Note.—Discussion open until April 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. WW 4, November, 1960.

[1] Div. Engr., Calif. State Div. of Small Craft Harbors, Sacramento 14, Calif.

action in 1957, it soon became aware of the fact that few engineering consultants, including many who specialized in waterfront construction, had had any prior experience in marina planning. The Division has since been assisting public entities in the development of public marinas, largely through contracts with consulting engineers, and has become acutely aware of the need for greater dissemination of ideas and principles for proper marina construction. In this paper these problems are discussed and the consensus of the Division staff is expressed concerning their proper solution.

SCOPE

Many of the elements of design involved in planning a small-craft marina are basically similar to those used in other types of construction. It is the purpose of this paper to present for consideration by the engineering profession only those criteria that are peculiar to small-craft-marina design problems, particularly to problems that have arisen in California. Many of the planning and design principles discussed apply equally to other regions, but it should be understood that the statements contained herein are derived from practice and observations within the one state only. Factors controlling the economic feasibility of marina construction are not discussed, and the relative economic merits of the various alternatives for component features are merely touched upon. Economic considerations could well be the subject for another paper. The information presented may, however, be of assistance in selecting types of design and materials best suited for a given set of conditions. The various suggestions offered should not be considered comprehensive, as many factors will be encountered in the planning of any large marina which may have been neglected or merely touched upon herein.

Most of the discussion herein concerns problems that have evoked constructive criticism throughout the state; problems which marina designers have found particularly challenging or controversial. Many of the views expressed are by no means the final answer, and represent only the findings and ideas of the staff of the Division of Small Craft Harbors as a result of 2½ yr of experience since it was organized in the winter of 1957-58.[2] During this time, however, the staff has reviewed plans for about 40 proposed public marinas and visited well over 100 existing public and privately financed and operated marinas throughout the state. Consequently, most of the views expressed come from experience with many cases of similar situations. It is significant that California's marina consultants disagree on many of the points raised, and it is hoped that this paper will elicit further constructive criticism on a nation-wide basis.

Design criteria for small craft harbors may logically be divided into two general categories: Functional arrangement of facilities and structural design. The former depends largely on the characteristics of the site and on market analyses which determine the types and quantities of facilities needed. Structural design is affected by exposure to corrosion, wave forces, surge,

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[2] "Planning and Development of California's Marinas," by J. W. Dunham, Civil Engineering, ASCE, Vol. 29, No. 9, September, 1959.

wind stress, substrata characteristics, strength of materials, and various applicable codes.

LAYOUT CRITERIA

The primary considerations in layout planning are the need for protection against wave action and entrance shoaling. The former is primarily important for ocean sites and large bodies of inland water where approaching waves are frequently large. The problem of entrance shoaling reaches significant proportions at river sites and in areas subject to extensive littoral drift.

Protection Against Large Waves.—The normal requirements for marinas with floating slips are reduction of wave height to 1 ft or less, and prevention of serious surge. This protection is normally accomplished by enclosure insofar as is possible by breakwaters, dikes, revetted moles, or utilization of an interior basin. In each case, the entrance must be made no wider than the demands of navigation traffic require and as narrow as the requirements for navigational safety permit.[3] Where enclosure is accomplished by breakwater construction, the entrance should be located at a point where the natural bottom is at least the project depth of the harbor and where it will cause minimum exposure of the protected area to approaching waves. Such a location may be readily apparent to anyone with a general knowledge of the wave regime of the area, but wave refraction diagrams will assist in determining the best entrance site.[4] Once the design waves have been determined for each important direction of wave approach at the entrance, diffraction diagrams should be prepared to analyze the attenuation of wave energy in the protected basin.[5,6] Such diagrams will readily indicate the areas where slips may be constructed or where additional interior protective structures or wave-absorption beaches are required to prevent the ingress of waves which, even though materially reduced by diffraction at the entrance, are still capable of causing damage.

Jetty-protected entrance channels leading to interior basins require the application of additional design criteria (Fig. 1). The jetties must be carried seaward to a point where the offshore contours indicate slightly greater depth than the design depth of the entrance. The entrance, of course, must be at least as deep as the interior basin, and the width is controlled not only by navigation requirements but also by the amount of tidal prism that it must pass. This involves consideration of entrance-channel velocities under maximum tide-range conditions, both as to bottom scour and the control-robbing effect on under-powered craft. An additional control for such entrances is that the depth be such that the design wave will not break in the entrance. This means that the depth must be at least 1.3 times the design wave height at maximum low tide. Where possible, the entrance channel should not be alined with the major wave-approach direction. If this is done, waves will be propagated into the interior basin with little reduction in wave height, causing serious surge and wave-reflection problems within the marina. If such an orientation cannot be avoided, it may be necessary to provide additional entrance protection

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3 "Waterfront and Harbor Facilities," by Bur. of Yards and Docks, U. S. Navy, Tech. Publication Navydocks TP-PW-8, September 1, 1954.

4 "Refraction and Diffraction Diagrams," by J. W. Dunham, Proceedings, First Conf. on Coastal Engrg., October, 1950.

5 "Generalized Wave Diffraction Diagrams," by J. W. Johnson, Proceedings, Second Conf. on Coastal Engrg., November, 1951.

6 "Offshore Breakwaters," by R. Silvester, Proceedings, ASCE, Vol. 83, No. WW 3, September, 1957.

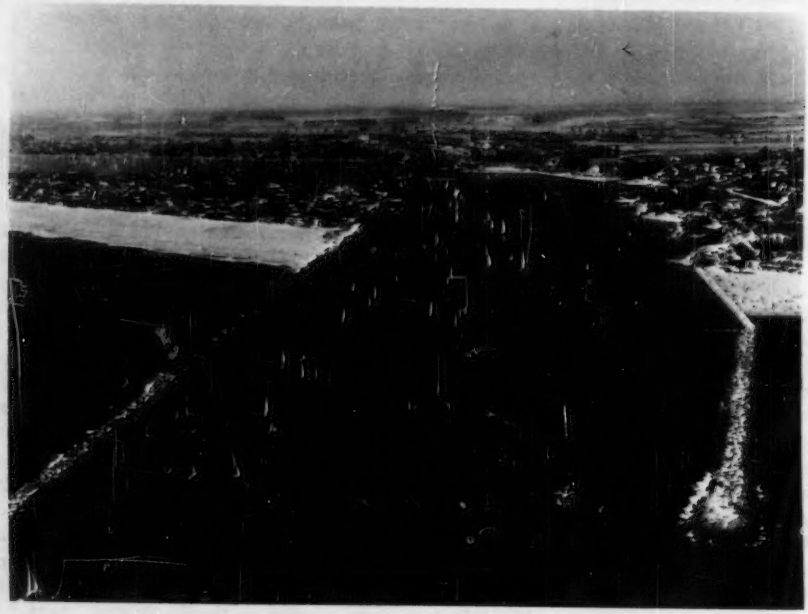

FIG. 1.—WIDE ENTRANCE CHANNEL

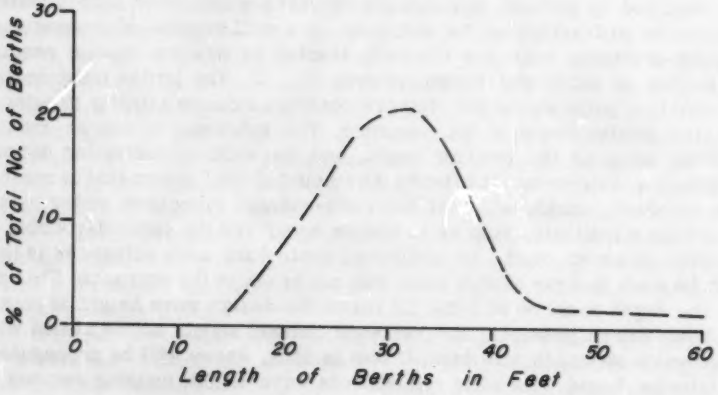

FIG. 2.—AVERAGE DISTRIBUTION OF SLIPS BY LENGTHS, SAN FRANCISCO BAY AREA

either through extending and hooking one of the jetties, or providing a detached breakwater off the end of the entrance channel. Such hooks or breakwaters must be so located as to avoid subjecting channel traffic to large quartering or beam waves as the vessels clear the entrance. In foggy areas such entrances can be very hazardous. If the marina is to accommodate sailing craft without auxiliary power, the entrance channel should be alined sufficiently off the prevailing wind direction to permit passage in or out on a single tack.

The determination of the design wave has been covered in other publications[7,8] and will not be discussed herein except to recommend that it be accomplished by specialists in this field if wave heights in excess of 6 ft have been observed in the area. The advancements in wave-prediction technology since World War II have been such that oceanographers and other scientists properly trained in this field can estimate probable maximum wave heights and directions in a given area with considerable accuracy. Such personnel are normally qualified to undertake the necessary refraction and diffraction studies for interior harbor design as well.

Internal Arrangement.—Several general principles apply in determining the proper layout of harbor facilities. The problem of wave interception by breakwaters and moles has already been touched upon, but in addition to this, the smaller or more fragile craft should be berthed in the more protected areas of the harbor from the standpoint of wave attenuation in the lee of such barriers. This normally means that the larger boats, which are more difficult to maneuver, are berthed near the entrance or near the outer ends of finger pier systems. The sailing craft also should be berthed near the entrance in slips so alined that they can be approached from a direction into the prevailing winds. This requirement, of course, does not apply to sailing craft that use auxiliary power within the harbor area. The distance betqeen slips in adjacent systems should be approximately twice the length of the longest slip for powered craft and $2\frac{1}{2}$ times this length for sailing craft without auxiliary power. This will provide adequate maneuver room for most types of craft. Finger pier systems should not be more than about 300 ft in length in order to hold the walking distance to the outer slips to a minimum. Marina patrons often carry heavy loads to their boats, and even with the use of carts that are frequently provided for this purpose, excessive lengths of walkways tend to discourage the marina pedestrians.

In planning the layout of slips, a careful analysis of probable boat sizes to be accommodated should be made. Each slip should be about 2 ft longer than the boat berthed, to permit the craft to be secured away from the deck structure without the stern extending into the channel. Fig. 2 shows the average distribution by lengths of slips in the various marinas of the San Francisco Bay Area. The size distribution will vary from place to place depending on the recreational habits and relative wealth of the marina patrons. The ratio of length to beam width is fairly standard for all powered craft and for all sailing craft. The sailing craft, of course, are much-narrower beamed than the powered craft. Slips should be wide enough to leave at least 2 ft in addition to beam width for each boat. The T-head at the end of a slip system extending out into the main basin may be used for berthing transient craft or craft

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[7] "Shore Protection Planning and Design," Beach Erosion Board, U. S. Army, Tech. Report No. 4, June, 1954, and Corrections, revisions and addenda, August, 1957.

[8] "Selection of Design Wave for Offshore Structures," by C. L. Bretschneider, Proceedings, ASCE, Vol. 84, No. WW 2, March, 1958.

larger than the largest slips of the marina will accommodate. As a rule-of-thumb for average size distribution, about 20 craft per acre can be conveniently berthed in the smaller marinas found in California. This ratio increases somewhat for marinas berthing more than 500 craft.

Parking areas should be carefully planned so as to reduce the walking distance to moored craft to a minimum. The use of parking moles and perimeter parking is advisable. By the same token, an adequate number of brows (hinged pedestrian ramps leading from the basin perimeter to the level of the floating system) must be provided for easy access to the slips. Fig. 3 shows a brow pier and brow leading to a floating boarding dock, used as a divider for a two-boat ramp. Piles through the dock are acceptable here where the entire dock perimeter must remain clear for boarding operations on a busy day. Piles through the exterior collars are preferable for floating-dock systems.

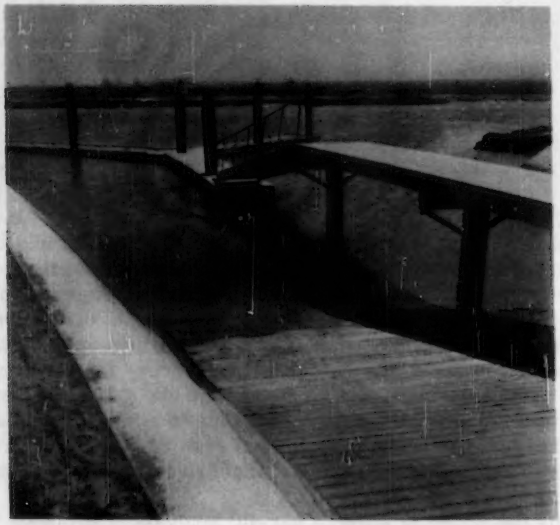

FIG. 3.—BROW PIER AND BROW

The launching area for portable boats should be kept as far from the berths for the permanently based fleet as possible, preferably near the harbor entrance. In some instances, a better solution has been to remove the launching area from the main basin of the marina altogether. The reason for this is that the portable-boat owner normally has interests and operating habits considerably different from those of the inboard and sailing-craft owners. Also the location of a portable-boat launching area well inside the marina often necessitates the passage of the launched craft past numerous berthed craft, adding to the traffic problem as well as causing wave damage to the latter. The harbor master's office should be located at a good vantage point for viewing all activities within the marina. It should preferably be near the entrance where incoming and outgoing craft may be monitored. Fueling facilities should be near the entrance but well protected from wave action and out of the way of the main stream of traffic.

Ancillary Facilities.—No marina is complete without the ancillary facilities required for the everyday needs of the average marina patron. These will differ in various areas, depending on the availability of similar facilities nearby, and on the recreational habits of the harbor patrons. These include restaurants, boat-repair facilities, boat sales and service area, dry storage for boats, dressing rooms and showers, a clubhouse, a restaurant, lodgings, and perhaps a recreational area, including a swimming pool or even a shopping center. Such facilities are usually required to help finance the non-revenue-producing structures.

The restaurant should be located so as to provide a good view of the harbor or the adjacent waterway. In some instances a location with a good view of water-sports activities will add to the attractiveness of the restaurant site. A good idea is to locate the restaurant at a water-ski take-off point or opposite a ski-jump ramp. In any event, the restaurant should have ample parking space for non-boating visitors who come only as spectators.

Boat-repair facilities should be located as far from the main entrance as possible, preferably in an area where boats can be retrieved without interfering with marina traffic. This is frequently the site of the larger launch hoists needed to get the heavier craft in and out of the water, whether for initial launching or for dry-dock purposes. Monorail launching is preferred because of the positive control afforded. Swinging booms or marine ways must be used, however, for handling craft with high masts. In some instances the repair facilities, marine ways, launching hoists, etc., border a nearby separate basin or another arm of the same waterway as that of the marina site.

Boat sales and service areas are often located adjacent to the repair yards, although this is not a necessity. Displays for boat sales are frequently found in some fairly prominent location on the marina perimeter which for functional reasons cannot be used for other purposes. Such displays, including auxiliary equipment and innovations in the boating field, frequently draw considerable interest and may comprise one of the main attractions of the marina as well as one of the more lucrative ventures connected with the overall endeavor.

Dry storage for boats is receiving progressively greater attention as existing marinas are filled and the need to conserve on sheltered water space becomes essential. Dry storage may be merely an open storage yard near a launching ramp or with easy access to one, or it may be an elaborate multi-storied shed arrangement involving the use of specially designed hoists and storage dollies. One such system now under design includes a double monorail system, which by alternately flaring the two monorails apart and bringing them together rotates the boat 90° wherever required. This system hoists the boat from the water, carries it to a series of lateral trolley ways on which the boats may be lowered and transported on dollies into sheds on either side of the monorail track at various levels. A less elaborate system employs large fork-lift trucks to lift the boats directly from the water and store them in racks.

Some marina operators keep a stock of rental boats for patrons who bring their own outboard engines. This type of operation is most successful where fishing is the primary attraction of the area. High-speed boating and water-skiing, on the other hand, require high performance craft, and the patrons of marinas and launching ramps near water areas used primarily for such sports usually provide their own boats.

Dressing rooms, showers, and rest-room facilities may be placed in any convenient location around the harbor perimeter, the exact site being determined largely by the location of the necessary utility lines that serve them.

The small craft marina is an ideal site for a clubhouse, which may be located in the marina compound or on a nearby vantage point. The requirement for a certain amount of exclusiveness from the activities of the general public dictates a site somewhat removed from main avenues of traffic, both vehicular and pedestrian. A site on an adjacent hill overlooking the harbor is ideal, if available. Clubhouse activities are not normally a direct part of the administration of the harbor operations, but are of a more private nature. The site may be reserved for operation on a land-leasing basis, or the clubhouse may be built as a part of the marina development and operated cooperatively by the patrons of the harbor.

STRUCTURAL DESIGN

In general, the latest concepts in structural design for the various facilities of the harbor should be used, provided they are not at variance with local codes controlling construction at the harbor site. C. A. Chaney has provided excellent material for basic design of various types of marina structures and their utility systems.[9] However, new materials and types of construction developed since 1950 should not be overlooked. The project life, as determined by financing requirements or other considerations, may sometimes affect the types of materials used. Shortage of funds may dictate the use of cheaper materials that must be replaced sooner. Also, where there are indications that the types of craft served by a given marina may change, as a result of changing conditions in the adjacent waterway or as a result of new boating habits in the area, certain elements of construction that would otherwise be made more permanent should be held to minimum cost to permit future modification. Greater use of modular construction will facilitate adaptation of harbor elements to the changing demands of the future. Fig. 4 shows a section of a modular floating slip system being delivered to a marina after launching and assembly elsewhere. Float and dock units of light-weight concrete poured monolithically are held together by timber rubbing strips bolted to the edge of the concrete decking.

Breakwaters and Jetties.—Where substrata bearing capacities permit, and where satisfactory quarries are within reasonable transport distance of the harbor site, stone is usually the best type of construction material for breakwaters and jetties. Along the open seacoast the trend in the past has been to use rubble mound construction with the stone sizes becoming progressively larger near and above the surface and toward the exposed side of a breakwater or end of a jetty.

The size of stone used in the armoured layer, from a depth about equal to the design wave height below maximum low water to the crown of the structure, is determined by the design wave. This design wave is determined by the oceanographic studies undertaken for use in layout planning. The modified Hudson formula is normally used in this determination.[10] A crown elevation

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9 "Marinas, Recommendations for Design, Construction and Maintenance," by C. A. Chaney, The Natl. Assn. of Engine and Boat Manufacturers, Inc., 1939, and Supplement thereto, 1947.

10 "Laboratory Investigation of Rubble-Mound Breakwaters," by R. Y. Hudson, Proceedings, ASCE, Part 1, Vol. 85, No. WW 3, September, 1959.

of 1.5 times the design wave height above maximum high tide will prevent passage of "green" water over the structure. Cost considerations normally dictate crown elevations lower than this: About +14 ft to +22 ft above mean lower low water along the California coast, where design wave heights range from 15 ft to 25 ft. It should be noted that much larger waves may break seaward of the structure. Hence, the maximum design wave possible is that which breaks at the structure, about 0.8 times the bottom depth at maximum high tide.[7]

The latest trend is toward the construction of a solid or near-solid core to prevent the propagation of long-period waves through the breakwater. Considerable surge and even regenerated wave action in the protected area has

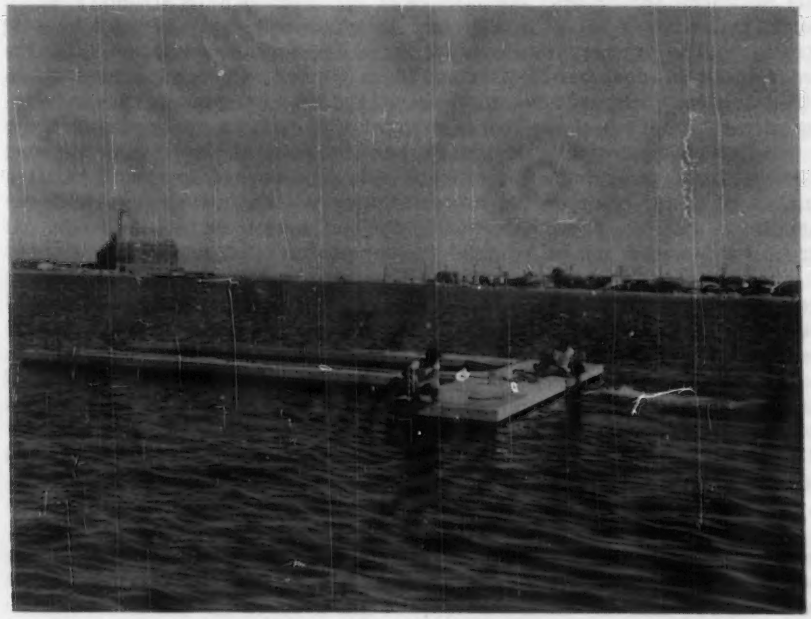

FIG. 4.—SECTION OF FLOATING SLIP SYSTEM BEING DELIVERED TO MARINA

resulted from the use of porous structures. The use of moles satisfies this requirement as does a breakwater with a relatively fine rock core graded to a massive stone facing. Such construction in jetties is required to prevent sand-flow through the structures where they cut through the breaker zone and beach face. Where the design wave requires stone in sizes larger than that available from local quarries or which are too difficult to handle with available construction equipment, concrete tetrapods[11] or other special shapes[12] are used in the armour layer. The proper constants used with the

11 "Breakwater at Crescent City, California," by J. E. Deignan, Proceedings, ASCE, Part 1, Vol. 85, No. WW 3, September, 1959.

12 "Breakwaters in the Hawaiian Islands," by R. Q. Palmer, Proceedings, ASCE, Vol. 86, No. WW 2, June, 1960.

Hudson formula[10] determine the weights of such units. Other types of construction materials in exposed reaches of the California coast, where good quarries are usually within economic transport distance, have not proved practical. However, in partially sheltered areas, timber and steel sheet-pile breakwaters have been used with some success. These are normally designed against clapotis wave action by use of the Sainflou formula[7] for the design wave at high tide. M. A. Gouda[13] has recently presented a new formula for calculation of hydrodynamic pressures on breakwaters. Shock pressures[14] are not normally taken into consideration, as these structures usually have adequate flexibility to absorb these extremely short-duration forces without damage. Considerable toe scour may be expected in front of such a breakwater. Dumped stone is normally used to stabilize the scoured areas as they develop. Pile breakwaters with openings left between the piles or between solid sheet pile segments will permit the passage of waves through the structure with a height reduction of much smaller proportion than the ratio of voids to solid structure. The primary requirement for a sheet-pile breakwater is that the brace piling be designed to withstand the maximum withdrawal forces where "A"-frame or "billboard" bracing is used, and the maximum cantilever bending moment where a single row of vertical piles is used. On one project, the entire frontal membrane is designed to be constructed of closely spaced round piling battered away from the exposed side, and braced by a much smaller number of batter piles on the protected side. Alternate shorter piles in the membrane will be driven to lesser depths butt down to reduce the voids. All structural timbers and piling used in such construction should be creosoted in accordance with American Wood Preservers Association standards.[15] Steel structures are normally protected against corrosion with cathodic protection, although a recent innovation in this field is the application of coal-tar epoxies for surface protection. In order to prevent over-topping, vertical-faced breakwaters should rise to twice the height of the design wave at high tide. This is seldom practical, however, and the usual compromise is a single wave height above high tide.[16] Lower heights may be used where over-topping will not disturb facilities immediately behind the breakwater. This results in a somewhat lower design force.

Much thought has been given to the development of floating breakwaters. In some of the smaller lakes and reservoirs in California, where long straight sections of log are readily available, log booms have been used with some success against the short-period waves generated in these small bodies of water. Against any significant wave action, however, the logs have little effect. One project now under design calls for a breakwater comprised of sections of large pipe, with ends sealed, supporting heavy diaphrams extending part way to the bottom. They will ride up and down with the tide but will be held against lateral movement by guide strips attached to pile dolphins. Exposure of this structure only to very short-period wind waves and the wakes of passing water craft suggests that it may be adequate for the project site. One advantage to be gained will be a good view of the interior basin from approaching small craft at all stages of tide, this is an important safety factor when traffic at

13 "Hydrodynamic Pressure on Breakwaters," by M. A. Gouda, Proceedings, ASCE, Vol. 86, No. WW 1, March, 1960.

14 "Shock Pressures Exerted by Breaking Waves on Breakwaters," by S. Nagai, Proceedings, ASCE, Vol. 86, No. WW 2, June, 1960.

15 "Manual of Recommended Practice," Amer. Wood Preservers' Assn.

16 "Model Study of a Hydraulic Breakwater Over a Reef," by C. M. Snyder, Proceedings, ASCE, Part 1, Vol. 85, No. WW 1, March, 1959.

the entrance is heavy. Wave tank experiments reported by R. L. Wiegel[17] indicate design requirements for such a structure. At Crescent City partial protection for an interior basin is provided by concrete baffles fastened between the pilings of a pier and extending to within a few feet of the bottom. This treatment has proved very effective.

For more exposed sites, the United States Navy has experimented with floating breakwaters formed by barges submerged at the seaward end to sufficient depth to intercept the bulk of the wave energy. These experiments have been generally unsuccessful because of anchorage problems and the structural requirements to withstand large wave forces. A new field of breakwater protection is the use of various types of pneumatic devices and hydraulic jets installed on the bottom designed to break up the orbital motion of the waves as they pass the line of installation.[10] Some of these devices have proved successful but costly to operate and maintain. It is probably that their use will be confined to navigation entrances between the ends of structural breakwaters.

Basin Perimeters.—In nearly all instances the basin perimeter requires some type of protection against erosion by sloughing or wave action. The simplest and cheapest protection is a revetted slope, usually about 1 on 3, the protection consisting of graded rock, sometimes placed on a gravel filter course. This type of perimeter treatment is wasteful of harbor space and should not be used where land costs are high or where the space available for marina development is limited. A better type of perimeter control is the vertical bulkhead, which may be a gravity concrete section or a concrete, timber, or steel sheet-pile cutoff wall with tiebacks. Bearing and shear strengths of substrata and availability of materials will normally dictate which type to use. Design criteria for such structures may be found in many textbooks and handbooks, and will not be discussed in this paper. However, a satisfactory compromise with the revetted slope treatment is to carry the bulkheading only to the approximate low-tide level and to revet the graded slope from that point part way to project depth in the harbor basin. When such treatment is used, the shallower draft vessels are berthed nearest the bulkhead where depths are limited by the sloping bottom.

Berthing System.—The berthing system in a marina may comprise fixed docks, floating docks, and anchorage areas, or any combination of these three. Fixed docks for small craft are not desirable where the normal tidal range exceeds four feet, or where any type of periodic or seasonal surface fluctuation in excess of this range occurs. In such instances, floating slips should be used. This being the predominant case in California marinas, primary consideration in this paper is given to the design of floating systems.

Anchorage areas are wasteful of space in a protected harbor, in that each anchor point must be sufficiently removed from all adjacent anchorage points to permit a 360° unobstructed swing of the anchored craft. Fig. 5 shows an anchorage area being replaced by a floating-slip system. Note should be taken of the piles at the ends of each slip pier, as well as along the walkway because of the strong surge, wind, and larger craft to be berthed. The larger craft are berthed on the leeward side of the main walk. Although bow and stern anchorage reduces the amount of waste space, it still requires the use of shore boats, and inefficient use of protected harbor area results.

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17 "Transmission of Waves Past a Rigid Vertical Thin Barrier," by R. L. Wiegel, Proceedings, ASCE, March, 1960.

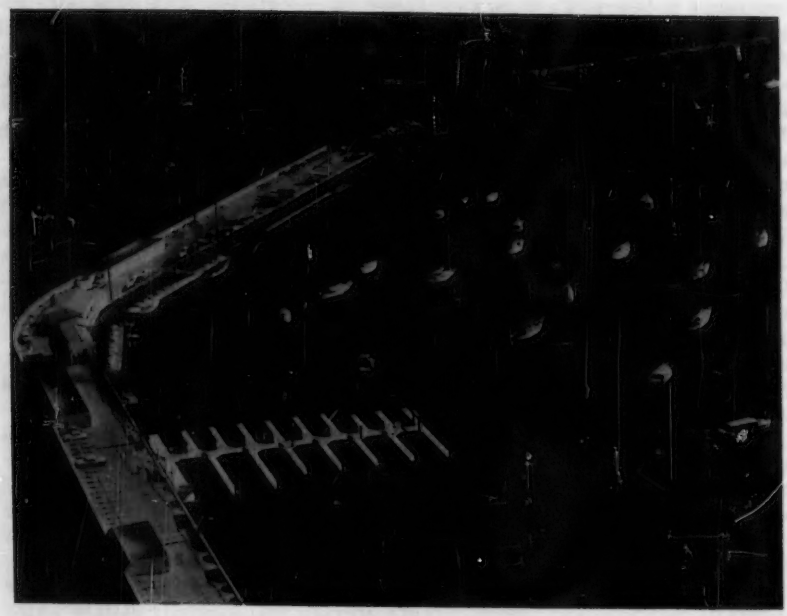

FIG. 5.—ANCHORAGE AREA BEING REPLACED BY FLOATING-SLIP SYSTEM

FIG. 6.—SIMPLE TYPE OF ANCHOR-PILE COLLAR

Any floating-slip system must be adequately anchored to withstand the lateral thrust of wind and surge on moored craft and on the system itself. Piling is normally used for this purpose where surface-level fluctuations are within reasonable limits. Anchor piles may be placed either at the ends of slip fingers or adjacent to the walkways. They should be adequate in number and so designed as to hold the system in position without bending failure considering each pile to contribute resistance to lateral movement proportionate to its stiffness modulus. This assumes, of course, that the piles are driven deep enough to prevent overturning in soft mud. The location of timber anchor piles at the ends of slip fingers is preferable if these fingers are sufficiently wide and have adequate strength built into their connections with the walkway. In this case, the entire system may be towed away for repair, relocation or replacement simply by removing the collars. In some marinas, the concrete piles are limited to interior locations where boats cannot collide with them.

Various types of slip collars have been designed, ranging from a simple U-collar made of wood or pipe, to systems of rollers running on ball bearings with metal rubbing strips on the piles. Experience has indicated that such rollers and rubbing strips are required only at two or three anchor piles in each independent slip system, these points to be determined by observation after the system has been installed. The remaining anchor piles normally float free of the collars except when sudden wind stress or current impulses cause sufficient bending of the key piles to bring the other piles into play. Such impulse periods are normally of short duration and do not result in significant abrasion of pile or collar. Fig. 6 shows a simple collar made by extending two transverse deck planks beyond the edge of the walkway.

Concrete piles, when properly constructed for the exposure to which they will be subjected, have a better appearance and greater project life than timber piles. A smaller number of concrete piles may be required because of their greater resistance to bending. Wood piles on the other hand are usually cheaper, and if properly treated with creosote preservative, will give many years of maintenance-free service. Pile anchorage has been used with surface fluctuations up to about 25 ft in small-craft slips adjacent to the Sacramento River. The greater ranges often encountered in reservoirs require some system for strutting the floating units out from shore or anchoring them in place with cables.

Each independent floating unit should be made as rigid as possible, to provide stability against wave action. Hinged connections between slip fingers and walkways should be avoided, and the strength of such connections should be developed with fillets to prevent horizontal deflection. Cantilever strength against vertical flexing at these joints should be built into the structure in any way possible. The latest trend is toward narrow decks to economize on cost. Walkways are seldom more than 8 ft wide and slip fingers more than 4 ft wide. Fig. 7 shows a simple, economical design that will provide adequate stability for well-protected marinas. One system, designed for Monterey Harbor, involves unit construction of opposing slip fingers together with a finger-width section of walk connected by short sections of walkway, the length of each section of walk being the width of the slips thus formed.

In some instances, the slip fingers are inclined to the walkway to facilitate ingress and egress by the moored craft or to give better alignment with the prevailing winds. This involves special design and additional cost, and is normally not worth the effort from either standpoint. Right-angle connections are preferred except in special cases. Hinged, non-torsion-transmitting connections between adjacent units and at angles in long sections of main walkway

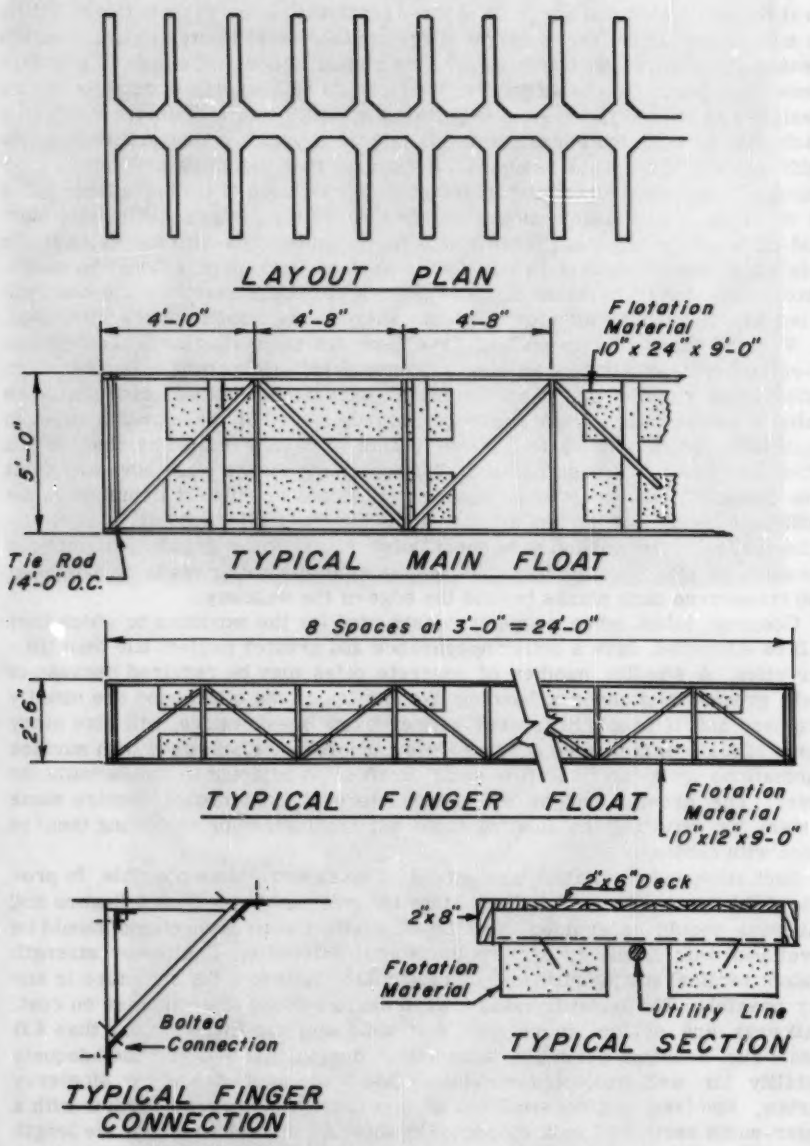

FIG. 7.—EXPERIMENTAL MINIMUM DESIGN, LOW COST, FLOATING DOCK SYSTEM

may be required where wave forces may be too great for transmission from one unit to the other.

Single-slip systems, in which each boat is flanked by a slip finger, are preferred by most boat owners for obvious reasons. However, double-slip systems, in which two boats are moored in each slip with a tie pile or a dividing strip in the center between the two boats, save the construction of one slip finger and approximately the width of a finger for every two boats berthed. Harbor patrons usually have definite feelings in this matter, some preferring the lower berthing fees charged at double-slip systems to the more costly luxury of having access to their boats from either side. Where tie piles are used to divide the double slips, sliding tie rods are usually provided. This system is more costly than the dividing-strip system in which a plank, pole or narrow finger float is extended out from the main walkway mid-way between the slip fingers. Dividing planks or poles may be cantilevered out from the walkway or may be hinged horizontally and supported by flotation. The latter system is preferable, in that cantilevered dividing strips sometimes become deflected below the water surface and constitute a hidden hazard to craft maneuvering into their berths.

Flotation.—Various types of floats have been used for floating berths, but the most successful appear to be those made of fiberglass-reinforced plastic, concrete, and various types of water-resistant synthetic materials.[18] Plywood floats coated with reinforced plastic have proved advantageous in supporting the excessive weight of covered docks where a high buoyancy ratio is required. They are also used where non-standard dimensions are required in a system comprised principally of concrete or plastic floats made in standardized molds or forms. Such floats are usually more expensive than other types, and consequently, are used only in special cases. Dual steel-tube floats bolted to a steel deck frame, and coated with a coal-tar epoxy are not under trial at one Sacramento River marina. Steel drums and even logs have been used for short-life installations, but are not recommended for permanent construction.

The first fiberglass-reinforced plastic floats used on the West Coast proved unsuccessful because of faulty manufacture. The process has now been refined by greatly increasing the molding pressures, application of additional glass reinforcement at points of stress or weakness, close inspection, and pressure testing, to the point where the incidence of failure has been reduced almost to zero. These floats, besides being exceptionally strong and durable, are completely resistant to sea water as well as to all types of contaminants. Because they are light in weight, they may be readily removed from the system by ballasting with water, submerging from under the deck framing, and then dry-docking for inspection and clean up. A current innovation in this field is a walkway unit with a non-skid fiberglass-plastic tread, pressure bonded to a plywood deck. The manufacturer expects to establish prices that will be competitive with timber decking.

Concrete floats have the advantage of inertia under choppy surface conditions, with little response to slight changes in pressure. A deck system resting on concrete floats generally gives the impression of greater stability than does a system supported on any other type of flotation in current use. Many failures have resulted from the use of concrete in floats largely because of unsatisfactory design, vibrating, curing, and launching. Because of their excess weight, special equipment must be used to place them in the water, and

[18] "Problems with Small Craft Harbors," by H. M. Noble, Proceedings, Sixth Conf. on Coastal Engrg., December, 1957.

unless care is exercised, they can be easily damaged. Transportation and launching costs are often higher than for other types of floats. Unless properly vibrated and cured, they are apt to develop cracks and slow leaks. The latest concept of proper concrete float design involves the use of steel forms, interior baffles, vibration with special small-shaft vibrators, no reinforcing, and steam curing. Final sealing of the cover with an epoxy bond after launching assures uniform temperature throughout float and cover units under site conditions. The floating units in Fig. 4 and 5 are made with light-weight aggregates. In both concrete and plastic float construction, the floats are not normally secured to the deck frame but merely held in place by a collar of decking material. Specially designed and manufactured concrete and plastic floats in standard sizes are roughly comparable in price under current market conditions.

Of the synthetic materials, extruded polystyrene "styrofoam" in logs or cut blocks have proved very popular for flotation on the Pacific Coast because of their relatively low cost and simplicity of installation. Because of its lightness and tendency to "pop out" from under the decking in rough water, this material is normally dowelled or strapped to the deck frame to prevent displacement. Although completely impervious to water and unaffected by most contaminants, polystyrene will deteriorate on contact with relatively small concentrations of gasoline or oil. In some instances, protective coatings have been used to prevent damage to this material. Such coatings usually make cleaning easier because of the smooth surface achieved. Cut sections of this material have a rough surface to which marine growth readily clings, resulting in a large accumulation of marine plant life in a short time. Although this growth in itself is not harmful, nor does it reduce the buoyance of the float, it is unsightly, and may harbor parasites or other living organisms that are harmful to the decking.

A new flotation product that has been on the market since 1958 consists of molded polystyrene, a low density, non-connecting cell structure that is molded under heat and pressure, each cell fusing with its neighbor to form a relatively tough, resilient, homogeneous product. The skin formed in the molding process gives a pleasing appearance, and the molds can be prepared in various shapes and sizes. Some of the early units suffered from lack of quality control, the cells failing to fuse properly, with resultant spalling off from the surface upon abrasion and breaking open under severe impact. Most of the units however, have remained intact and unaffected for a year or longer, indicating that their use for flotation of docks will be long lasting and equally as satisfactory as the extruded product. Efforts are now being made to determine the requirements for proper quality control of this material, and when properly specified in construction constracts and inspected for conformity thereto, it may prove to be a popular type of flotation product.

Polyurethane has most of the characteristics of polystyrene, and in addition, may readily be poured into any shape desired at the site of the work. It is more expensive, but unlike polystyrene, is unaffected by hydrocarbons. None of these light-weight plastic materials, however, have the permanence of concrete or plastics in locations exposed to relatively strong wave action. Because of their inherent lack of structural strength, they may be readily cut or mutilated and fall easy prey to vandalism. Their use should be confined to well protected and adequately monitored locations.

Deck Framing.—The principal function of the deck structure, besides supporting pedestrians, is to transfer loads from one float to another and to hold the entire deck unit as stable as possible. Accordingly, joints must be made

as rigid as possible, lap joints being preferable to butt joints. Wherever possible, flexure should be limited to the main timbers between points of support or joints. Wherever joints start to "work," deterioration will follow rapidly. A large part of the structural strength of the system is usually incorporated in the rubbing strips. Accordingly, these rubbing strips are normally made of 2 x 8's to 2 x 12's, sometimes in double thickness with joints lapped, and held securely in position by diagonals and cross-backing. However, in order to avoid excessive wind stress, and to keep the deck low enough for convenient access to small boats, the deck height should be kept to within about 15 in. above the water surface, which in turn limits the width of rubbing strip which may be used without permitting frequent immersion in the water. Timber framing and decking should be constructed with salt-treated lumber, in accordance with specifications of the American Wood Preservers Institute. Such treatment makes painting unnecessary, although this is sometimes preferable for appearance. On occasion, a sand-paint coating has been applied to the deck to effect a non-skid surface. This, however, is not too successful because it is difficult to clean and tends to become discolored by spilled liquids and ground-in solids.

One of the tendencies of timber decking is to twist and cant when not held horizontal in firm support for any considerable distance. This tendency can be overcome by corrective ballasting of the floats or prevented by increasing the width of walkways and fingers. The walkways should normally be not less than 5 ft and the fingers not less than 3 ft in width without special provisions to prevent warping. The concentration of buoyancy forces near the edges of the deck frame will also assist in maintaining horizontal stability. With polystyrene and similar material this is easy to do by placing the material in two rows at either edge of the float, leaving the middle unsupported. In floating walkways where fingers are not used, utility lines under the decking should be centered to prevent listing. Corrective ballasting of any type of hollow float may be accomplished by placing sand inside the float on one side or the other to overcome listing. Maintenance personnel should look for incipient listing in new construction, and take corrective action before the wood structure takes a permanent set.

A few integral float and deck units such as those shown in Fig. 4 and 5 have been placed on the market, with some degree of success. Where such units have failed, the failure has usually occurred because the nuts embedded for bolted connections were not properly bonded to the main reinforcement of the float. Substantial rubbing strips bolted to the sides of the floating units are required to transmit stresses between units.

Where timber decking is used, anything less than nominal 2-in. thickness has proved unsatisfactory. Diagonal decking is sometimes preferable to longitudinal or transverse placement because of the structural advantages gained. The cost of doing this is but slightly greater, and the resultant solidity of the structure may be worth the additional effort. Deck planks should be fastened to the framework with galvanized nails, and 1/4 in. spaces should be left between adjacent planks. All bolts and other hardware used in exposed places should be galvanized. In some instances the anchor piling is driven through holes left in the decking for that purpose. This practice is not considered desirable because of the encroachment on the deck space and because of the difficulty of removing any section of the floating system from the anchorage if this becomes necessary. The finishing touches comprise application of accordion-folded fire-hose or rubber fenders to the rubbing strips, attachment of mooring cleats, installation of utility outlets, etc.

Covered Berths.—Most of the berths along the coast are open, the tenant boat owners covering their craft as necessary with tailored cloth or plastic covers when not in use for any length of time. In the hot interior areas, however, covered storage is preferred, more as a guard against the hot rays of the sun than against precipitation. Covered storage may take the form of pole-supported sheds, or the entire system may be floated, depending largely on the amount of water-level fluctuation contemplated. Sheds supported on poles must give ample clearance at maximum water-surface elevation for ingress and egress of boats. Very frequently, however, the side sheathing is carried only to the high water surface elevation. Where the entire shed is floated, the supporting floats, of course, must have ample buoyance to support the load (Fig. 8). Corrugated aluminum roof and side sheathing is preferred because of its light weight, corrosion-resistant qualities, and ability to reflect the sun's rays with little heat absorption. The principal objection to covered storage is

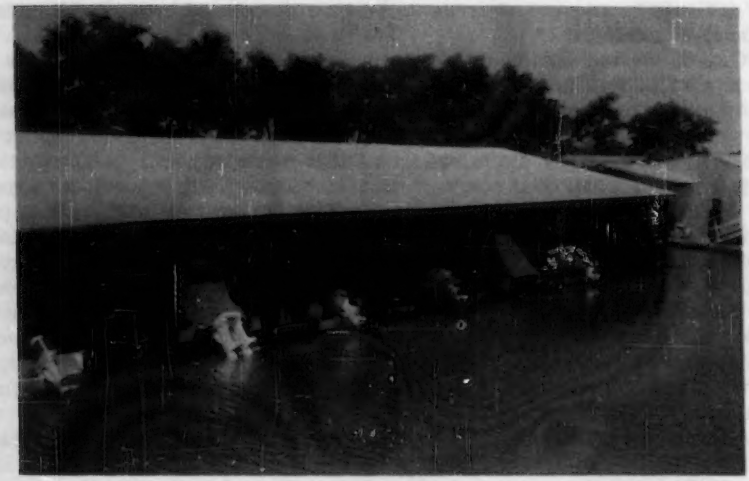

FIG. 8.—FLOATING COVERED BERTH NEAR STOCKTON, CALIFORNIA

its lack of aesthetic appeal. Most people enjoy seeing the craft exposed in their berths. Where covered systems are installed, they should be designed for an entire floating system. The practice of allowing tenants to construct their own sheds over leased slips usually results in an unsightly collection of different shapes and sizes, conforming to the various tenants' tastes and financing capabilities. Relatively little effort has been devoted to the architectural treatment of covered storage. In view of the great strides that have been made in the architectural design of other structures, it would appear that this field should receive considerably more attention in the future. Because protection against direct sunlight rather than precipitation is the primary requirement, large suspended, light-weight, latticed covers over entire marinas may be a practical solution.

Approach Ramps, Utilities, and Miscellaneous Provisions.—The floating system is connected to shore by a brow leading from the basin perimeter to

the floating deck level. Various brow designs have been used with varying degrees of success. Any simple truss or girder system that results in relatively light construction is satisfactory for this purpose. Such a brow is shown in Fig. 3. Anodized aluminum is slightly more expensive, but has a pleasing appearance and gives years of maintenance-free life. It should be noted, that the dead weight of the hinged ramp on the floating system requires additional buoyance in the area of contact. Utilities placed on the floating system should include fresh water, with a hose bibb at about every other slip; electric power, with an outlet at about the same interval; and fire-fighting equipment for every 15 or 20 slips. Flood lighting should be provided if possible by a few lights set high above the marina. The stringing of electric wiring along the piling with an occasional dropcord light is unsightly and should be avoided. A public-address system that will reach all parts of the marina is important. Gear boxes should be provided at each slip if possible. A convenient place for gear storage is at a fillet between a finger pier and the walkway.

The past trend toward high supporting capacities for extensive floating systems has proved to be unnecessary where reliable flotation is incorporated in the design. Considerable savings in construction cost may be effected without danger to the public by increasing the stability and decreasing the buoyance. The deck framing and flotation for such systems may be designed for dead load plus a minimum of 25psf live load at complete submergence of floats, unless the local code is more severe. This will provide more than ample flotation for almost any foreseeable circumstance at the marina. The introduction of a percentage-of-submergence factor only complicates the computation without contributing an additional safety factor. In some instances a large congregation of persons on the floating system may exceed the 25-psf limit, but the incipient sinking of the system normally tends to disperse the crowds before submergence occurs. Relatively narrow decks tend to prevent such crowding. Many floating deck systems in California marinas have given years of safe service with far less buoyancy than the suggested minimum. Small floating docks that tip easily under eccentric loading require greater buoyance.

Elimination of Fingers.—In order to save space and simplify design, slip fingers have been omitted in some instances and boats docked along side the main walkway in one of three ways. The usual way is to pull the boat up along side, mooring to cleats bow and stern. This procedure is most common on rivers where the flow is predominately in one direction. Walkways, sometimes two or three abreast, trail downstream from an anchored access walk leading out into the stream perpendicular to shore. A space of about four boat widths is left between the walkways to permit boats to maneuver up along side from the trailing ends. Where the streams carry heavy debris during winter floods, the entire system must be removed during the flood season. In any event, the entire structure, including the flotation units, must be of exceptionally sturdy construction to withstand the impact of floating debris.

In some instances, where the water remains relatively calm, the craft are berthed with their axes at right angles to the walkway. Special bow clamps are provided when the craft are moored heading into the dock. This type of berthing is objectionable because of the resultant abrasion from the craft moving with relation to the dock under the influence of wave action. Also the boats can be boarded only stepping over the bow, a rather difficult and precarious maneuver with some types of craft.

Boats can also be moored bow out with special long hooks hinged horizontally that fasten into eyes at each corner of the stern of each boat, holding

it out a sufficient distance from the rubbing strip to clear the outboard engine or propeller. This method of berthing eliminates abrasion but it not too successful in rough water. In this case the craft must be boarded from the stern. The hooks must be so secured into the eyes that they cannot be dislodged by wave action tending to roll the craft on its axis. In general, bow or stern berthing should be used only for relatively small craft.

Fuel Dock.—Some local codes require that all fueling facilities be on solid ground or fixed piers. However, any relatively stable floating dock is to be preferred for fueling small craft, as boats being fueled tend to rise and fall with the float in unison with the wave action. Such installations can be made relatively safe by keeping fuel storage tanks ashore, providing check valves to prevent siphoning in case of line breaks, using substantial flexible hose, properly guarded, and taking advantage of all safety-measure possibilities offered by the peculiarities of the site. The larger fueling stations are usually installed by the fuel companies. Most floating fuel docks in California have excellent safety records. All nearby electric motors, switches, etc. must be explosion proof, however, and adequate fire-protection equipment must be at hand.

Launching Ramps.—Launching ramps should preferably be constructed of concrete, either poured-in-place or precast, placed on a gravel bed. They should be broomed or groomed horizontally to provide a non-skid surface. Boarding docks should be provided (Fig. 3). Use of raised dividing strips to segregate launching lanes is generally discouraged because of the tendency of inexperienced trailer operators to cross dividing strips and foul two lanes instead of one. It should be noted that 600 boats have been launched and retrieved in a single day from the 150-ft multiple launching ramp shown in Fig. 9. The slope of the ramp should be between 12% and 15% and should continue to at least 3 ft below the surface at maximum low water. If the water surface elevation is expected to change appreciably from year to year, some type of portable ramp construction should be used. Reinforced concrete planks fastened side by side have been used successfully for such purposes (Fig. 10). Where the waterway being served has mineral or other contaminants of any kind, fresh-water flushing tanks should be provided for the convenience of the portable boat operators. Hoses and fresh water under pressure should also be provided to wash contaminants from parts of vehicles that have been submerged in the launching process. Ample maneuver area should be provided at the ramp, and separate approach and exit roads should be provided if possible.

CONCLUSIONS

Marinas for small craft should be so protected as to reduce surge to a minimum and assure that the maximum wave height at any of the floating slips does not exceed one foot.

2. Design considerations for layout planning should include:

(a) locating the entrance where the natural bottom is at least project depth,

(b) planning the entrance to reduce shoaling to a minimum and yet provide for adequate navigational safety,

(c) meeting the special needs of sailing craft,

(d) providing adequate and convenient vehicle parking areas,

(e) separating portable craft activities from the berthing area,

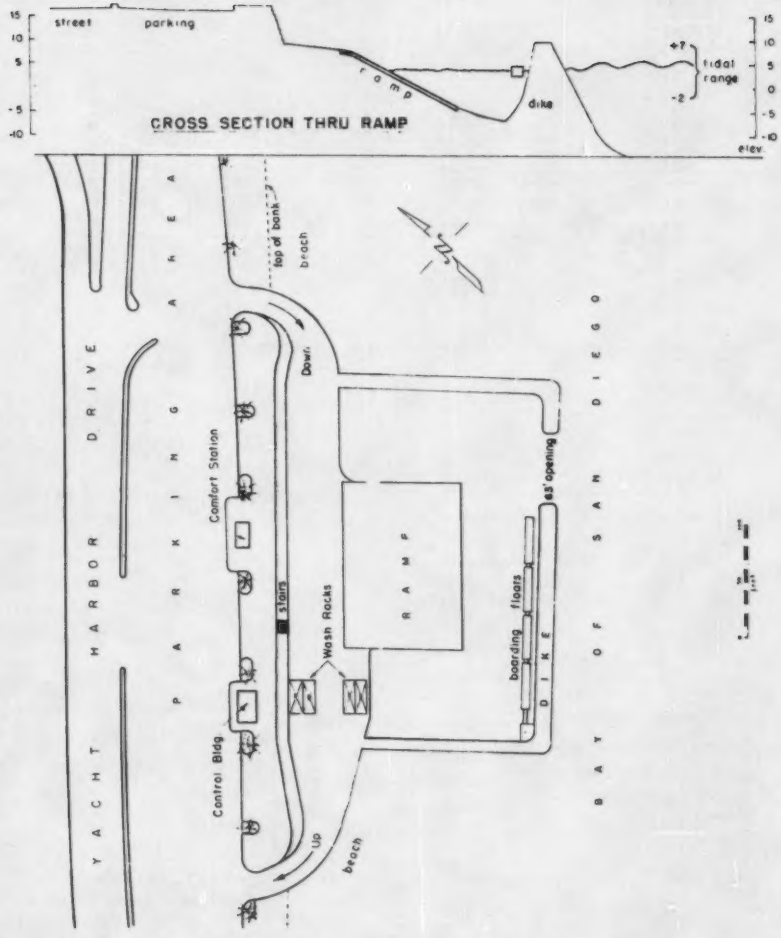

FIG. 9.—MULTIPLE LAUNCHING RAMP

(f) placing the fuel dock in a safe, convenient location,
(g) making efficient use of available harbor space in the layout of slips,
(h) accurately estimating the sizes of berths required.

3. Ancillary facilities should be provided to meet all the requirements of the prospective marina patron including restaurants, boat repair facilities, boat sales and service areas, dry storage for boats, dressing rooms and showers, a clubhouse, a restaurant, lodgings, and recreational facilities.

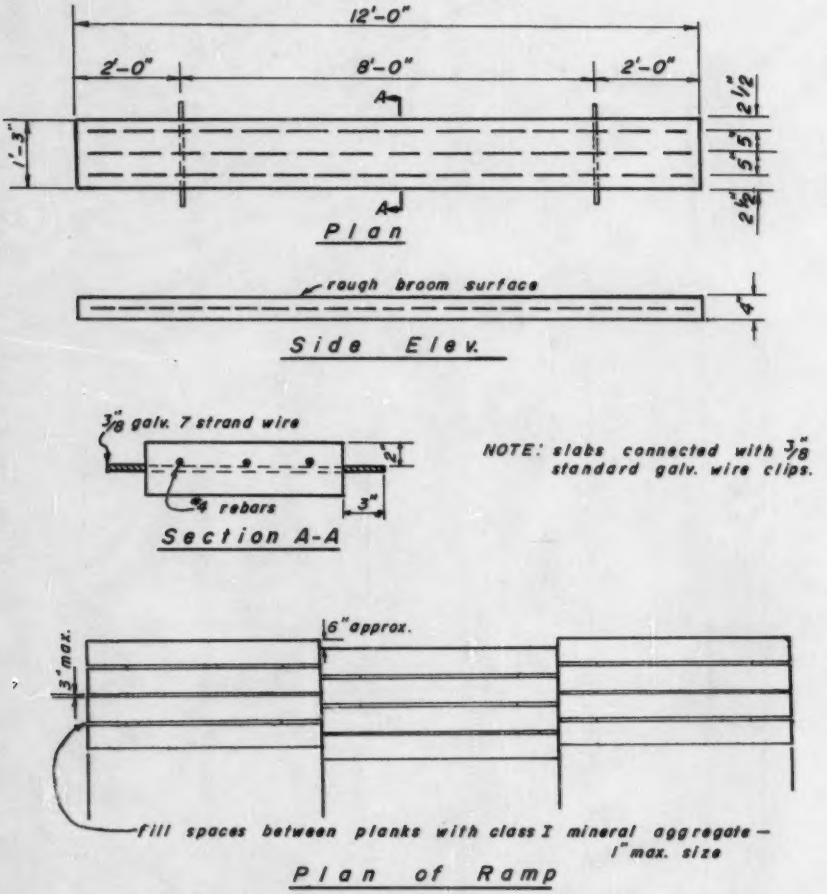

FIG. 10.—PRECAST CONCRETE PLANKS

4. Design formulas for breakwaters of various types derived by Hudson, Sainflou, Gouda, and Wiegel may be used for marina protective structures.

5. Rubble mound breakwaters subject to long-period wave attack should be relatively impervious.

6. Partial vertical barriers, floating or fixed, may provide effective protection against boat wakes and short-period wind waves.

7. Hydraulic or pneumatic breakwaters may provide additional protection at exposed entrances to coastal marinas. Prototype testing of such devices is an indicated need.

8. Vertical bulkheads for basin perimeters prevent waste of space where the area of a marina site is limited or expensive.

9. Anchorage of boats in a marina is inconvenient and wasteful of a protected basin space. Fixed berthing slips should be provided where normal water-level fluctuations are less than about 4 ft. Floating slips are required where this range is exceeded.

10. Floating slip systems may be fixed in place by piles where water-level fluctuations do not exceed about 25 ft. In some reservoirs anchorage and strutting out from shore may be necessary.

11. In a floating system fixed by piles, only two or three piles are in continuous contact with the collars and only they need rollers.

12. Relatively narrow decks with strong, rigid connections between fingers and walkways provide both economy and stability in a floating-slip system. Weak connections soon lead to deterioration of the entire system.

13. Single and double-slip systems both have strong proponents. A possible compromise may be the substitution of a very narrow finger float hinged horizontally in lieu of a tie pile for the divider between the two berths of a double-slip system.

14. Right-angle finger connections are preferred to inclined connections.

15. Hinged, non-torsion-transmitting connections should be used to couple adjacent floating systems where differential wave-response requires.

16. Floats for floating berths made of concrete and fibreglass-reinforced plastics give promise of greatest life if constructed properly. The inertia of concrete floats tend to give greater stability under wave agitation, but are more difficult to launch and recover for inspection or servicing. Modular concrete float and deck units give promise of low maintenance cost and ease of modification of slip arrangement. Steel-tube floats may have greater strength for resisting the impact of floating debris in rivers, but they should be covered with an epoxy coating to prevent corrosion. Extruded polystyrene is a lighter, cheaper, long-lasting flotation material whose buoyance is not destroyed by puncture. Molded polystyrene is cheaper than the extruded product, more resilient and adaptable, but may deteriorate unless manufactured properly. Polyurethane has all the characteristics of polystyrene but unlike it, is unaffected by hydrocarbons. These homogeneous substances lack the structural strength of other types of flotation and may fall prey to vandalism. Oil drums, logs and other makeshift floats have shorter life and are not recommended for permanent installations.

17. Deck heights of floating berth systems should be kept to about 15 inches above the water surface.

18. The tendency of timber decking to twist and cant often can be overcome by corrective ballasting of the floats. Concentration of flotation elements near the edges of a floating dock increases stability and may help prevent listing.

19. Timber rubbing strips for floating docks should be substantial and designed to transmit much of the stress between floating elements under agitation by wave action.

20. To assure long life anchor piles and other fixed structural timbers should be pressure-treated with coal-tar preservaties. All other wood construction should be protected with salt preservatives.

21. Timber decking should be at least of nominal two-inch thickness and applied diagonally where extra structural strength is required.

22. Covered berths are often demanded by marina patrons. The covering may be either pile-supported sheds, usually sheathed with corrugated aluminum. More professional thought should be given to the design of such structures.

23. Utilities lines serving floating systems should be centered under the decks walkways not stabilized by slip fingers. Utilities should include fresh water, electric power, fire-fighting equipment, adequate lighting and a public address system. Gear boxes on or near the slips are desirable.

24. A trend toward lighter but stronger floating dock construction is noted. The minimum live loading may be reduced to 25 psf if local codes permit.

25. Floating docks that trail downstream are used successfully on rivers where basins behind the river banks are not available.

26. Berthing with bow clamps or stern hooks can be successful only in very calm water and with small boats.

27. Floating fuel docks are safe and practical if installed properly in the right locations.

28. Launching ramps should be of concrete construction where possible, with a 12% to 15% slope. Raised dividing strips should not be used on wide ramps. Boarding docks, fresh tap water, ample maneuver space, and nearby parking area should be provided at the launching site.

29. The various consultants in the marina-design field have been working out their problems independently in most cases, often disagreeing with each other. They should pool their talents in a concerted effort to provide the public with marina planning of a caliber more in keeping with the traditions of the profession. Such an effort is warranted by the magnitude of the current boom in recreational boating.

ACKNOWLEDGMENT

The writer wishes to express appreciation to Patrick J. Creegan of the firm of Creegan and D'Angelo for his helpful comments, and to the personnel of the engineering section of the Division of Small Craft Harbors for their assistance. Special acknowledgment is made to John S. Habel and A. B. DeJarnett for their comments and helpful review of the manuscript while in preparation, to Louis Stradiotto for preparing the drawings, and to Beverly McDevitt for typing the original manuscript.

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DISCUSSION

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A BREASTING DOLPHIN FOR BERTHING SUPERTANKERS[a]

Closure by John M. Weis and Virgil Blancato

JOHN M. WEIS,[1] A.M. ASCE, and VIRGIL BLANCATO,[2] F. ASCE.—The writers are sincerely grateful to Messrs. W. F. Manning, Nicholas Bilonok, and Jerome M. Richolson for their discussions in connection with the design of a breasting dolphin for berthing super-tankers.

The discussion by Mr. W. F. Manning is of particular value, for the authors had completely overlooked, and neglected the forces of friction between the fender and the vessel at the points of contact. As pointed out by Mr. Manning, these forces materially affect the energy absorption capacity of the structure, and should be taken into consideration in the computations. Inasmuch as previous tests to determine the energy absorption capacity of the retractable fender system were conducted with static loads acting on an installation having only a maximum of retraction of only 4 in., the forces of friction between the fender and the applied force were unimportant. It is regrettable that these significant forces were overlooked in the application of the retractable fender principal to the breasting dolphin design, with the 20 in. retraction. Mr. Manning is to be commended for his thorough analysis and for his valuable contribution to the design.

Mr. Manning is entirely correct in his computations, in that the force P required to retract the fender becomes approximately 3.5 W for an A = of 45°, considering the coefficient of friction between the ship's hull and the fender timbers to be 0.25, and maintaining the value of 0.3 for the coefficient of friction of steel on steel as originally presented by the authors. Substituting this value for P in equations 1.07 and 1.08, the horizontal and vertical components acting on the cylinders of the dolphin will then be 2.7 W.

Rather than maintaining the original fender weight, and encountering an undesirable fender pressure on the ship's hull, as considered by Mr. Manning, the authors would prefer to reduce the fender weight by eliminating the precast concrete blocks. Because the only purpose of these blocks in the original design was to provide weight, their elimination is desirable, and greatly facilitates field construction. This would reduce the weight of each fender section from 37 kips to 21 kips.

Thus, the force required to move the six 20 ft sections up on the brackets equals 6(3.5W) or 440 kips, and the resulting energy absorption of the six fender units equals

$$E_{RF} = (440)\ (20) = 8800 \text{ in.-kips} = 4400 \text{ in.-tons}$$

Under the 440 kips load the cylinders would deflect approximately 14 in. at full retraction of the fenders.

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[a] September, 1959, by John M. Weis and Virgil Blancato.

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With the fenders retracted the full 20 in. and the dolphin cylinders each deflected the maximum permissible 22 in., the total energy absorbed by the structure, would be approximately 7,240 in.-tons. This would result in an approximate hull pressure of only 5,400 lbs per lineal ft as compared to the 6,000 lbs per lineal ft allowed in the section on design criteria.

Mr. Manning's comment that the 24 WF 120 walers with a span of 30 ft would be overstressed is correct. However, it was the intention of the authors that the deck system on the structure would serve the dual function of bracing the wales. In this regard, deck beams located at the third points between the cylinders and serving as cross-bracing for the wales permit a uniform load of 7.1 kips per lineal ft of wale.

Mr. Bilonok's presentation of the cofferdam-type dolphin structure, used by the U. S. Army Corps of Engineers on the Delaware River for rehandling stations for the hopper dredge Comber and the sump rehandler New Orleans, is extremely interesting. The discussion of the forces considered in the design of this dolphin is believed to be most appropriate, and is an excellent example of the type of investigation that should be conducted prior to the design of any structure. However, inasmuch as the dolphin consists of a 25.5-ft-diam sheet pile cell filled with sand and gravel, the writers do not believe it should be compared with a flexible dolphin. The berthing forces on this semi-rigid structure must be absorbed by the fender, and protection for the ship's hull must be provided for entirely within the fender system.

The authors considered a rubber buffer and timber piles fender system similar to that mentioned by Mr. Bilonok, for incorporation in their flexible dolphin structure design, but rejected the scheme for the following reasons. The cylindrical form of the dolphin would offer only a limited tangential surface of contact between the fendering and the vessel. Under these conditions the large berthing forces imposed by the tankers, would overstress the timber and the rubber at the points of contact, resulting in frequent breakage of piles and cracking of rubber. The main advantage of the retractable fender in conjunction with the flexible cylinders is the large area of contact obtained between the berthing vessel and the dolphin.

Mr. Richolson's question regarding the possible substitution of timbers for the structural steel members in the dolphin design is considered to be most pertinent.

The frame units of the fender could be fabricated entirely with timbers—using 14 by 14 in. timbers in place of the 12 WF 106 vertical posts and four 14 by 14 in. timber wales in place of the three 24 I 120 sections. However, it should be taken into consideration that the buoyancy of the submerged section of an entirely timber fender will act adversely on the effective weight of the units. This effect should be considered in any design incorporating all timber construction of the fender units. In the event additional weight is required to achieve an optimum ratio of energy absorbing capacity between the fenders and the dolphin cylinders, concrete blocks may be added.

It is not considered advisable to use timber frames in place of the 24 WF 120 sections to support the brackets and to connect the cylinders. During an actual berthing, each cylinder would be subjected to varying amounts of torsion and deflection. As a result the frame connecting them would be subjected to considerable shearing stresses that would undoubtedly result in damaged timber members.

In closing, it is considered appropriate to reiterate that in presenting this paper of flexible dolphin design, it was not the intent of the authors to present a complete structural design, ready for fabrication and erection. Instead, it

was the intention to present a principal of design in which a flexible supporting structure would work together with a resilient fender to absorb berthing forces of large ships in exposed locations with maximum protection for the vessel. The details of application of this principal are left to the individual designer who, availing himself of site conditions, can design a structure suitable to the size of the ships scheduled to use the facility and provided with the transfer equipment required.

A CASE OF CRITICAL SURGING OF A MOORED SHIP[a]

Closure by Basil W. Wilson

BASIL W. WILSON,[13] F. ASCE.—The author is greatly appreciative of the very detailed discussion of Mr. Joosting, which clearly shows a most incisive reading of the author's paper. In replying to the discussion it seems appropriate to deal with the points as brought out by Mr. Joosting.

(a) The rather simplified representation of the transverse seiches existing at the end of the long rectangular basin, given in the author's Fig. 2 and Table I, is queried by Mr. Joosting. It is indeed correct that the two dimensional character of the rectangular dock has an important influence on the periods and properties of the possible modes of oscillation that can exist. Eq. 1 should properly be replaced by

$$\tau_{m,n} = \frac{2\,B}{\sqrt{g\,d}} \left[\frac{1}{m^2 + \gamma^2 n^2}\right]^{\frac{1}{2}} \quad \ldots\ldots\ldots \quad (34)$$

in which $\gamma = B/\lambda$ is the breadth-length ratio for the dock, while m and n(= 0, 1, 2, 3, . . .) are integers defining the co-existing harmonic modes or number of nodes in the transverse and longitudinal directions respectively of the rectangular basin. As mentioned by Mr. Joosting the author had indeed presented this formula in the cited paper,[1] although he had actually given it first in a still earlier paper[14] in which he had tabulated values of the periods $\tau_{m,n}$ for different values of m and n. These agreed generally with the values given by Mr. Joosting in his Table I except that in the latter the specified number of nodes along and across the basin would appear to require interchanging. Thus for n = 1 and m = 4, with $\gamma \simeq 1/3$ for the basin in question, Eq. 34 shows very readily that the period $\tau_{4,1}$ would be approximately 1/4 of 110 secs or 27.5 secs and not 69 sec, as given by Mr. Joosting. On the other hand for n = 4 and m = 1, the period $\tau_{1,4}$ would be 0.625 x 110 or 69 sec. The author seriously questions whether all the periods apparently observed by Mr. Joosting in his model were modes in accordance with the nodal combinations given in his Table I. The same values in essence evolve more readily from the nodal combinations given in Table 9.

The modes given by the author in Table I and Fig. 2 are the harmonic sequence of possible pure transverse seiches for the case of n = 0 in Eq. 34, and were adopted because they are believed to be reasonably representative of the regime of oscillations most likely to exist at the end of the dock where the ship was berthed. One good reason for saying this is that the entrance to the

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[a] December, 1959, by Basil W. Wilson.

[13] Prof., Engrg. Oceanography, A. & M. College of Texas, College Station, Tex.

[14] "The Mechanism of Seiches in Table Bay Harbor, Cape Town," by B. W. Wilson, Proceedings, 4th Conf., Coastal Engrg., Chicago, Ill. November, 1953; Council Wave Research, Berkeley, Calif., pp. 52–78.

dock happens to be located in the long side of the basin on the seaward side close to the basin end (≃900 ft) of our interest so that activating pulsations of wave energy entering the dock are directed transversely across the dock with maximum strength at this end of the long basin. The end of the dock nearest the entrance thus tends to behave partly as a quasi-basin in the transverse sense and its responses can be expected to be strongest to modes of oscillation in which the emphasis is on m rather than n in Eq. 34. Thus, assuming that an exciting frequency of from 54 sec to 57 sec were imposed on the dock, the writer's belief is that the response at the basin end in question would be of the m = 2, n = 1 category rather than the m = 1, n = 5 combination of Table 9 and would not differ greatly from an m = 2, n = 0 type oscillation across the end of the dock.

It will readily be apparent from Eq. 34 that with $\gamma = 1/3$ the influence of n in the factor $\left[m^2 + \gamma^2 n^2 \right]^{-\frac{1}{2}}$ is but little over 10% of that of m, so that if n also is less than m, as in the right-hand columns of Table 9, the term $\gamma^2 n^2$ is comparatively small compared with m^2. No very serious error is made therefore by taking n = 0 in these cases as was done in Table I. As a specific example we note that the two-dimensional oscillation m = 5, n = 4 has a period $\tau_{5,4}$ = 21.3 sec, but if n = 0, $\tau_{5,0}$ = 21.9 sec, which is not greatly different. The oscillation in this case is essentially the 5th harmonic of the fundamental transverse oscillation specified in the author's Fig. 2 and Table 1.

TABLE 9.—PERIODS OF TWO-DIMENSIONAL SEICHES IN HARBOR BASIN

Number of Nodes	along (n)	1	2	3	4	5	1	3	2	3	4
	across (m)	1	1	1	1	1	2	2	3	4	5
Period $\tau_{m,n}$ (in secs.)		105	92	78	69	57	54	49	35	27	21

It will be seen from Table 9 that the periods of the two-dimensional oscillations given in the last three columns all accord reasonably well with the trinodal, quadrinodal and quinquinodal seiches portrayed in Fig. 2. The sectinodal and septuanodal oscillations could equally well be justified. Moreover the periods in the first four columns of figures of Table 9, although varied, all result from what is essentially a uninodal oscillation (m = 1) at the basin-end in question. Consequently, the general system of pure oscillations given in the author's paper is largely validated as applicable to our problem.

(b) Mr. Joosting maintains that his model tests have shown that the wave height in respect of the 110 sec oscillation is about 40% greater in the corner of the basin than at the location of the tide-gage, which is approximately 670 ft from the corner along the landward long side of the basin. This seems rather remarkable insofar as the 110 sec period applies to an m = 1, n = 0 type of two-dimensional seiche in which amplitudes should tend to be the same all along the long sides of the basin. Even if the oscillation accorded with a two-dimensional seiche of m = 1, n = 1 type, such as would give a period $\tau_{1,1}$ of 105 or 106 secs according to Mr. Joosting's Table 1 or Table 9, the theoretical indications are that the height in the corner would exeed that at the tide gage by only 6%. At any rate the adoption of the tide-gage values of amplitude for the fundamental seiche of 110 sec period is completely justified by the assumption of an m = 1, n = 0 type of oscillation.

Mr. Joosting quotes South African and French model tests on the basin in question as showing that amplitudes of oscillations within the basin are directly proportional to the wave periods for constant input wave height at the wave generator at the location of the harbor approaches. His conclusion that a ratio of 1 : 0.5 : 0.2 would have been preferable to the author's 1:0.75 : 0.55 for the relative amplitudes of the fundamental, second and fifth harmonic seiches. This argument would apply if in fact the heights of the relevant long-period waves entering the approaches to the harbor were actually equal. There seems no special reason, however, for making such an assumption of equality. The author's ratios are based upon the concept of a wave spectrum in which spectral height tends to decrease with declining frequency. Since the period of the quinquinodal seiche (22 sec) is not far removed from the frequency band of visible storm waves, it is rational to expect that the height of long waves of this period would have been relatively greater than the height of long waves of 110 sec period. The same agrument would apply in respect of long-waves of 55 sec period. The concept of a wave spectrum in itself is entirely rational for it is known that the surge phenomenon always accompanies visible waves and swells reaching the afflicted harbor. For the particular occasion discussed (May 5, 1952) it has already been shown[15] that waves of 10 sec to 12 sec period with a possible height of 14 ft were probably prevalent in the far outer roadstead during the occurrence of surging. The author's amplitude ratios are admitedly conjectural, in the sense that the assumed wave spectrum itself is conjectural, but they are considered to be more realistic than the ratios advocated by Mr. Joosting.

(c) The author is grateful to Mr. Joosting for pointing up a drafting error in the designations of the curves of Fig. 4. The most right-hand curve should bear the numbers (1) and (2) while the third curve from the right should carry the number (3). Mr. Joosting's query regarding the relative tightnesses of the mooring ropes cannot be answered with any degree of certainty. It is the author's recollection that among rope failures reported for the harbor in question the heaviest toll was in fore and back springs, suggestive of the likelihood that these ropes, in the positions shown in Fig. 1, are often overloaded in normal berthing operations on ships.

(d) The author will admit that Mr. Joosting's suggestion of a quantity $H' (= -\zeta/\sigma)$ given in his Eq. 16(a) would give the presentation of the equations added meaning. The quantity ζ used by the author, however, is not without physical meaning since it can be considered to be the effective surge velocity of the water causing motion of the ship, operative over the length of the ship in its particular location.

(e) Mr. Joosting's belief that another factor, capable of taking account of the proximity of the ship to the quay and the sea-bed, should enter into Eq. (16) may possibly be correct. The factor $(\sinh k\,d - \sinh k\,s)/\cosh k\,d$, however, already considers some part of the influence of the water depth and the clearance s of the ship bottom above the sea bed. Also, the influence of the nearness of the quay would seem to be governable already to some extent in the equations by adoption of an appropriate value of virtual mass coefficient C_M, if we accept the assertion of Sommet[16] that its value of 1.1 to 1.2 for a

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15 "Research and Model Studies on Range Action in Table Bay Harbor, Cape Town," (reply to discussion), Transactions, South African Inst., Civ. Engrg., Vol. 2(5), May, 1960, pp. 89-96.

16 "The Troubled Waters of Table Bay Harbor," (discussion), by J. Sommet, Transactions, South African Inst., Civ. Engrg., Vol. 2(7), July, 1960, pp. 146-147.

ship surging in open water should be increased to from 1.2 to 1.4 for a ship flanking a quay.

(f) Mr. Joosting challenges the adequacy of the author's factor $(\sin k\,l)/kl$ in establishing the influence of the ship's length on the effective driving forces promoting movement of the ship, and claims that the formula given by his Eq. 1 is more correct insofar as it takes into account both the acceleration of the water and the buoyant forces on the ship of the water within the wave slope. The author and Mr. Joosting have crossed swords mildly on this issue elsewhere[17,18] but while the author admits that his version neglects the pitching motion of the vessel, he is unable to satisfy himself in regard to the complete accuracy of Mr. Joosting's formula owing to the lack of any published presentation of its derivation. Thus Mr. Joosting's Eq. 1 does not appear to agree with his original version of it[5] and his later publication,[19] although stating it in the form of Eq. 1, provides no derivation.

The competing factors are plotted by Mr. Joosting in his Fig. 2 and are seen to be quite similar in trend and not too vastly dissimilar in value, which suggests that the pitching effect may not be too important. But before accepting Mr. Joosting's claim that he is necessarily right and the author wrong, let us examine the case cited ($k\,l = \pi$) for which $(\sin k\,l)/k\,l$ is zero and for which Mr. Joosting's factor C_1 is 0.245 according to his Fig. 2. As Mr. Joosting rightly observes, this condition implies equality between ship length $2\,l$ and wave length and he argues therefore that the ship must necessarily respond to the wave slope in the manner of his Fig. 1. This, however, is wishful thinking because the position of the ship can be anything whatever depending on the period of the seiche in relation to the natural pitching period of the ship. For example, in the average water depth of 45 ft the period of seiche whose wave length would equal the length of the ship (350 ft) would be 9.2 sec. The free pitching period of the ship, however, could be expected to be of the same order (9), so that pitching resonance would result in the ship lagging the water movement by about 90° of phase angle. The ship would thus actually be horizontal for the situation shown in Mr. Joosting's Fig. 1. In consequence the effective driving force on the ship would be nil because the center half of the ship would be subject to a positive wave slope and the remaining end quarters to an equal and opposing negative slope. In these circumstances the author's factor $(\sin k\,l)/k\,l$ would be right and Mr. Joosting's formula wrong. The likelihood of pitching resonance occuring at $k\,l = \pi$, in depths of water normal to most harbors, is difficult to escape for all but the largest sizes of commercial vessels, so that the author's allowance for the influence of ship length would at least seem to be a safe approximation to the truth.

(h,i) The author would agree here with Mr. Joosting except insofar as the latter's Eq. 1 may still be a bone of contention.

(j) The author will concede that a solution to the differential Eq. 12, leads naturally to the form of Mr. Joosting's Eq. 2 rather than the author's Eq. 21 and is preferable because Eq. 12 applies to a seiche of the form.

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17 "The Troubled Waters of Table Bay Harbor," (discussion), by B. W. Wilson, Transactions, South African Inst., Civ. Engrg., Vol. 2(7), July, 1960, pp. 143-144.

18 "The Troubled Waters of Table Bay Harbor," (reply to discussion), by W. C. Q. Joosting, Transactions, South African Inst., Civ. Engrg., Vol. 2(7), July, 1960, pp. 147-150.

19 "The Troubled Waters of Table Bay Harbor," by W. C. Q. Joosting, Transactions, South African Inst., Civ. Engrg., Vol. 1(10), October, 1959, pp. 211-222.

$$\eta = A \cos \frac{m \pi x}{B} \sin (\sigma t + \epsilon) \quad \ldots\ldots\ldots\ldots (35)$$

rather than Eq. 15, which, inadvertantly, was incorrectly stated.

The author's Eqs. 22 are not invalidated by this error, although a correction must be noted in Eq. 22(ii) for the phase angle, as acknowledged under (l) below. The phase angles were not evaluated numerically for the problem of the paper and do not feature in any of its conclusions.

(k) Mr. Joosting's version of the author's Eq. 22(i) certainly provides a nice way of looking at it. In his Eq. 6 Mr. Joosting has further refined the equation into a very interesting and acceptable form.

(l) The author is pleased that Mr. Joosting has pointed up what was essentially a transcript error in the value of tan α in Eq. 22(ii). This should read in accordance with Mr. Joosting's Eq. (4). If the solution $X = X_1 \sin (\phi - \alpha)$ is used, however, the phase angle will be correctly given by

$$\tan \alpha = \frac{2 \beta(\sigma/\omega)}{X^{n-1} \Delta(n) - (\sigma/\omega)^2} \quad \ldots\ldots\ldots\ldots (36)$$

rather than Mr. Joosting's Eq. (5). For $n = 1$ Eq. 36 reduces to the well-known phase relationship, $\tan \alpha = 2 \beta(\sigma/\omega)/[1 - (\sigma/\omega)^2]$, for a linear damped oscillating system.

(m) On the basis of the foregoing replies to the various points made by Mr. Joosting, the author sees no reason to accept Mr. Joosting's contention that the effective amplitude of the fifth harmonic of the fundamental seiche was necessarily inadequate for exciting the critical ship motion. Mr. Joosting has put forward an interesting counter-presentation of the possible causes of trouble, but the author, while not disputing the possibility that coincidental seiches of longer period could also have been responsible for the breaking adrift of the ship, must reaffirm his conclusions as presented in his paper. The fact that other ships, moored in positions very close to that of the tanker in question, have responded to similar sea disturbances by surging at periods of from 20 to 30 sec has been established by actual measurement, in proof of which Mr. Joosting is referred to Table 2 in the author's paper,[20] which gave the results of measurements made during 1944-46.

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20 "Research and Model Studies on Range Action in Table Bay Harbor, Cape Town," by B. W. Wilson, Transactions, South African Inst., Civ. Engrg., Vol. 1(6 and 7), June, July, 1959, pp. 131-148, 153-177.

WAVES IN NAVIGATION CANALS DUE TO LOCK FILLING[a]

Discussion by J. Th. Thijsse, W. D. Baines, and H. J. Leutheusser

J. TH. THIJSSE,[6]—Mr. Partenscky has had the privilege of comparing theoretical studies with model experiments as well as with measurements in a "full-scale" canal. The result of both proves to be interesting.

In the model, the effect of a transition from a wide canal to a narrower section was compared with Eq. 13 for a gradual transition. This formula is not widely known, and as far as I know it has never been checked by experiments. The Karlsruhe tests show a deepening of the waves that is less than Eq. 13 predicts (Figs. 20 and 21). This may be due to the fact that the transition in the model is not perfectly gradual. An abrupt transition will deepen the surge to a lesser degree than a gradual one. In this regard compare Eqs. 13 and 9. It would be interesting to make a series of experiments, starting with an abrupt narrowing (or widening), and making the transition zone longer in every succeeding run.

In the prototype measurements, an interesting item is the distortion of the surge caused by the fact that the deeper part (the trough) is propagated slower than the beginning of the phenomenon, where the canal still has its original depth. In the paper (Eq. 3 b), the retarding factor contains the coefficient $\frac{3}{4}$. In the past there have been discussions about this coefficient. De Saint Venant's equation, for vertical walls, leads to another value, viz $\frac{3}{2}$. Measurements in Dutch canals (1934 Twentekanal; 1936 Julianakanaal) have yielded results that were very close to de Saint Venant's equation.[7] Now, the observations in the Dortmund-Ems Kanal (Fig. 24) show a reduction of speed at the trough of the surge that is decidedly less than de Saint Venant indicates, and that is close to Eq. 3 b. This controversy has to be explained. Possibly, observations in other canals may help in this respect.

Friction has not been taken into account in either equation. This may be a reason for the disparities. A computation with the method of characteristics would be necessary in order to discover what really happens. When designing a canal with locks, it may be necessary to know which steepness the front of a positive surge or the rear of a negative one may eventually acquire. Thus the problem also has a practical aspect.

W. D. BAINES,[8] ASCE and H. J. LEUTHEUSSER,[9]—It is of interest to learn through this paper of the present research activities in one of Europe's oldest hydraulic laboratories. Design of navigation locks and experimental investi-

[a] March, 1960, by Hans-Werner Partenscky.

[6] Former Dir. Hydr. Lab., Delft, Holland.

[7] Proceedings, International Navigation Congress, Section Inland Navigation, Paper No. 5, 1935.

[8] Assoc. Prof., Dept. of Mech. Engrg., Univ. of Toronto, Toronto, Ontario.

[9] Lecturer, Dept. of Mech. Engrg., Univ. of Toronto, Toronto, Ontario.

gations of open channel surges caused by power house operations have for many years intrigued the members of the Karlsruhe institution. The author has shown ingenuity in combining the techniques of both fields to produce this useful report.

Unfortunately, the paper is somewhat lacking in clarity in some sections due to differences in terminology and expression between Europe and America. However, these should not deter the practising engineer in putting the material to use, particularly because most of the equations are classical ones and can be derived from first principles.

Among the terms which the writers found confusing is the correction factor, given the symbol μ. It is the discharge coefficient for the filling system and is a function only of the geometry of the inlet passage as controlled by the gate opening. The discharge coefficient is readily determined from model tests and the variation, if large, can be taken into consideration in the computation of the filling curve; the integration of the head versus time equation is performed for a series of steps within which the discharge coefficient is assumed constant. In the same paragraph the author also uses the term "filling section" for what is commonly referred to as the area of valve or gate opening.

The writers note, however, that Eqs. 28 and 29 are not complete without stating the underlying assumptions; that is, (a) the gate is still opening when the level in the lock rises above the sill, and (b) the gate opening is small as compared to the depth of water in the upper pool. The first condition obviously restricts the applicability of these equations to locks of relatively low lift. When filling through the lock gate augments an initial inflow through a deep-lying entrance (as in the case of the Meppen lock facilities mentioned toward the end of the paper) a different set of equations will apply.

The one-dimensional assumptions used to describe the wave characteristics result in a really simple and readily understandable set of equations. The neglect of friction and the constancy of wave velocity are good assumptions as long as the reach is short and the surges small. If these conditions are not met then a more complete solution such as the method of characteristics must be used. The writers have found that a good discussion of open channel surges and related computation methods has been presented by Jaeger.[10] This also contains an extensive bibliography on the subject.

It can be noted that the equations for the open channel surges are analogous to the simple equations used to describe water hammer in a piping system. In each case small wave fronts are assumed to travel at constant speed along the canal or pipe. For water hammer the change in pressure head is the primary wave characteristic and is analogous to the change in level in the canal. Furthermore, it can be shown that corresponding to the author's Eq. 1 the continuity equation for water hammer waves requires that

$$dz = \frac{dQ}{g\,A/c} \qquad \ldots\ldots\ldots\ldots\ldots\ldots \quad (50)$$

in which A is the cross section area of the pipe, c denotes velocity of water hammer wave, and g refers to the acceleration due to gravity. Thus the quantity of ωW in open channel surges corresponds to $g\,A/c$ for water hammer waves. Using this substitution and the water hammer equations found in many text books one can directly obtain the relationships for transmitted and reflected waves at changes in channel cross section, dead ends, and reservoirs

10 "Engineering Fluid Mechanics," by Ch. Jaeger, Blackie and Son., London, England, 1956.

as given by the author. In addition, one can quickly obtain relationships for other geometrical conditions such as the junction of two canals. It is admitted that such relationships could also be easily obtained from first principles.

The greatest value of the analogy to water hammer is that it may lead to schemes by which the size of the surges can be reduced in a canal. For example, if there exists a dead-end lateral canal connected to the main canal at some point the size of the surge passing the lateral will be modified by the transmission of a surge up the lateral, reflection from the dead end and retransmission at the junction. If the lateral has the same depth as the canal then

$$\frac{z_3}{z_1} = \frac{1 + 3\,W_2/W_1}{(1 + W_2/2\,W_1)^2} \qquad \ldots\ldots\ldots\ldots (51)$$

in which z_1 is the height of surge in the main canal approaching the junction, z_3 denotes height of surge in the main canal leaving the junction, W_2 refers to width of lateral canal, and W_1 is the width of main canal.

Study of this equation reveals that if the lateral is much wider than the canal the surge is materially reduced in size. For example, if the ratio of widths is 4 the surge is reduced to 78% of its initial size, and if the ratio is 8 the reduction is to 36%. In most cases, the amount of excavation necessary for an effective lateral would be excessive, but there are locations where the natural topography would provide for it. Existing natural pools may sometimes be considered as wide laterals and the effects on the surge determined roughly by this type of analysis.

Another scheme used to reduce the size of water hammer pressures is the pressure relief valve. The analogy of this for open channel surges is the weir mounted parallel to the lock. A small steady flow is maintained through the canal (often for sanitary reasons) which spills over the weir. When the lock is filled this flow is diverted and the negative surge is correspondingly reduced. This scheme is even more effective in the reduction of the positive surge resulting from emptying of a lock upstream.

According to the author's concluding remarks, no real need for controlling the surge size exists at the present moment. However, this situation will change one day and simple analyses as discussed herein may prove useful in directing the hydraulic engineer toward a satisfactory solution.

BREAKWATERS IN THE HAWAIIAN ISLANDS[a]

Discussion by Robert E. Hickson and J. E. Deignan

ROBERT E. HICKSON,[5] F. ASCE.—The composition and design of breakwaters and jetties in exposed locations is a subject that has engaged the attention and interest of engineers since earliest historic times. They are still struggling with the problems involved, and the author has presented much discussion and experience data that should be very helpful. Additions to the store of knowledge and theoretical treatment of the problems involved, by engineers experienced in such works, are much needed.

This discussion is based primarily on experience in design, construction, and maintenance of rubble stone jetties on the North Pacific Coast of the United States. Some experiences are related to show the behavior of heavy concrete blocks where exposed to severe wave action. It is hoped these experiences will furnish some useful information to add to the data on the subject.

Breakwaters and jetties basically serve different purposes, but in many situations are subject to the same destructive action of heavy seas. The maintenance of breakwaters at an effective height is more essential, especially on smaller harbors, than is the case with jetties; the primary purpose of jetties is to direct and control currents and channel depths, unless, as in many cases, they serve also as breakwaters.

The principal parameters affecting the stability or permanence of breakwaters or jetties constructed in exposed waters are: (a) finished slopes, (b) height of structures as related to wave heights and tidal variation, (c) wave heights, (d) nature and weight of components of construction and manner of placing, (e) angle of wave attack, (f) terminal structure, and (g) nature and condition of foundation materials, and so on. These parameters will be discussed.

With regard to item (a), it is recognized that flatter slopes are the more stable. But reduction of slope does not provide a complete answer. On the parts of structures above low water level, there are many examples in experience that even very heavy components are moved on practically level surfaces down to, and to some extent below, low water level. Fig. 19 shows the complete removal of all stone (45% of which averaged 10 tons with many weighing 20 tons) from top of the old Columbia south jetty enrockment, in a period of 20 yr (1913-1933). Fig. 20 also shows flattening on the sea side of enrockment of the reconstructed jetty superstructure in about 24 yr, 1936 to 1960. The extent of erosion shown is estimated from aerial photographs of 1960.

As to underwater slope in areas where prevailing rough seas will not permit the use of floating plant, it must be realized that the construction on flat

[a] June, 1960, by Robert Q. Palmer.

[5] Cons. Engr., Portland, Ore., Formerly, Chf. of Engrg. Div., Portland, Ore. Dist., and Chf. of Constr. and Operations of North Pacific Div.

slopes in deep water is impracticable, and material has to be dropped or dumped to take the angle of repose, with equipment operating off the top of the structure. No planned rearrangement is possible.

The height of a rubble breakwater structure is very important, if disintegration is to be avoided. No matter what the components of construction, or how well capping on the sea side may be placed, if the seas frequently break over the top in force, material will be carried over and the back slope will also deteriorate. When this takes place, a breakthrough of the crest will occur. The breakthrough will then widen progressively in both directions, and the result, in time, will be a general lowering.

With a flat slope carried to such height that energy of the breaking wave will be spent before the crest is reached, the chances of securing a stable structure would be greatly enhanced. It must be realized, however, that with waves of the higher ranges this criterion would result in a structure so large as to be economically impracticable.

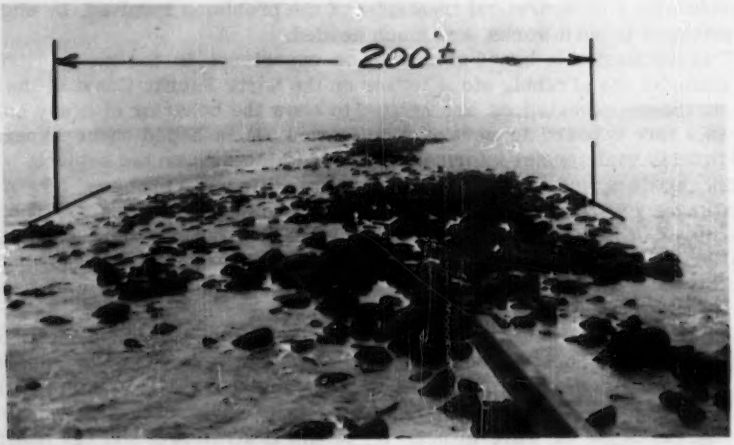

FIG. 19.—TOP OF ORIGINAL ENROCKMENT AFTER 20 YR, 1913-33. ADJACENT DEPTHS ABOUT 40 FT.

The height of waves to be designed for is, of course, a basic consideration. Unfortunately, reliable data covering wave heights on various parts of the coast line are very limited. More scientific investigation and records along this line would be of great assistance in design.

The weight of individual components of construction, whether stone or concrete, necessary to secure a stable structure, has been the subject of many discussions and many empirical formulas have been devised for solution of the problem.

The writer has used heavier pieces of stone for the lower wave heights, even up to 15 ft, than indicated by the formula developed by the Corps of Engineers Waterways Experiment Station, and it has been found from experience with large concrete blocks cast in place, that for waves of 30 ft to 40 ft trough to crest, the derived weights, while very large, are not as great as those of some cast blocks of concrete that have been moved, even on a level base.

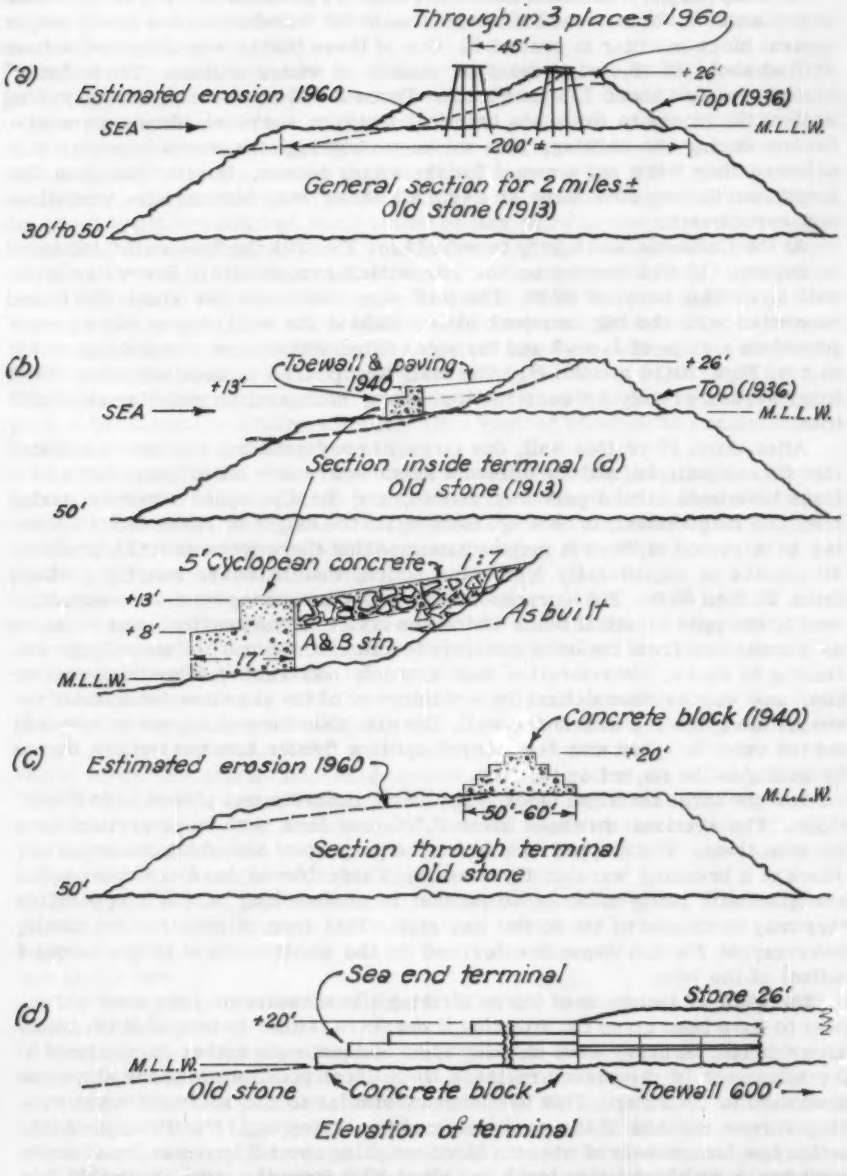
(a)
Through in 3 places 1960
45'
+26'
Estimated erosion 1960
Top (1936)
SEA
M.L.L.W.
200'±
General section for 2 miles±
Old stone (1913)
30' to 50'
(b)
Toewall & paving
1940
+13'
+26'
SEA
Top (1936)
M.L.L.W.
Section inside terminal (d)
Old stone (1913)
50'
5' Cyclopean concrete 1:7
+13'
+8'
As built
A&B stn.
17'
M.L.L.W.
(c)
Concrete block (1940)
Estimated erosion 1960
+20'
M.L.L.W.
50'-60'
Section through terminal
Old stone
50'
(d)
Sea end terminal
Stone 26'
+20'
M.L.L.W.
Old stone
Concrete block
Toewall 600'
Elevation of terminal
COLUMBIA RIVER SOUTH JETTY — OUTER 2 MILES
SCALE
0'
60'
120'

FIG. 20

At Umpqua jetty terminal concrete blocks 15 ft-by-15-ft-by-8 ft were built at low water level on a rock fill base around the foundation of a much larger central block in order to protect it. One of these blocks was observed to have shifted about 20 ft during the first season of winter storms. These "wing" blocks weighed about 135 tons each. Three 2-1/2 in. (stock) chains joining each of the blocks to the main terminal block to serve as hinges were also broken during the shifting. It is not known how high the waves were, but it is believed they were not unusual for the winter season. Similar blocks at Columbia north jetty also were shifted. All these wing blocks were poured on new enrockment.

At the Columbia south jetty terminal (see Fig. 20) the "toe wall," indicated in section (b) was poured on the old settled enrockment at low water level, with an outside berm of 50 ft. The wall was continuous for about 600 ft and connected with the big terminal block. Behind the wall, heavy stones were placed on a slope of 1-on-7 and the voids filled with concrete to a depth of 4 ft to 5 ft, Figs. 20(b) and 20(d). The wall was poured in sections about 30 ft long, separated only by construction joints, each section weighing about 360 tons.

After about 20 yr, this wall, due largely to undermining, has been separated into its components, and the different sections are now out of line. Some sections have been turned part way around, and the Cyclopean concrete paving has, to a large extent, broken up. Here again the height of waves is not known, but in a period of 20 yr it may be assumed that there were, in total, probably 40 months of significantly high wave action, that is waves ranging perhaps from 25 ft to 40 ft. The purpose of this wall-and-paving type of construction next to the jetty terminal block where sea action is the heaviest, was to serve as a transition from the solid concrete terminal to the rubble stone jetty extending to shore. Deterioration was scarcely noticeable for the first year or two, and was evidenced first by a picking out of the old stone from under the blocks along the sea face of the wall. Cavities thus formed appear to have admitted water to enter with "explosive" uplift or "water hammer" effect, thereby adding to the impact on the face.

For the large terminal block, Fig. 20(c), concrete was placed in four sections. The sections averaged about 2,500 tons each, with large vertical keys between them. The stepped face is for the purpose of absorbing the impact or shock of a breaking wave in increments. These blocks have not moved, but are gradually being undermined so that in another long period it is possible they may be caused to tip to the sea side. This type of construction would, however, be far too expensive for use on the whole exposed length (about 4 miles) of the jetty.

The angle of incidence of waves striking a breakwater or jetty does not appear to have been given the attention it deserves, either in formulas or model experiments. A heavy wave striking at an oblique angle rather than normal to the alignment of the structure tends to roll components along the slope and downward to the water. This is somewhat similar to the action of seas running across the end of the structure, and in overtopping. Verification of this action has been observed where a stone weighing about 5 tons was found on the sand beach at about $\frac{1}{2}$ tide level, but about 50 ft from the jetty. Note that this occurred in the surf zone of breakers. It is believed, and is evident, that much of such destructive action takes place farther to sea, beyond the surf, where the wave action is much more effective in moving stone. Somewhat similar action is understood to be taking place at the Crescent City, California break-

water where heavy tetrapods were placed on the sea slope. This action is considered to be more powerful than the backwash of seas striking normal to the sea slope, and then receding.

Foundation conditions are of vital importance in any design of a new jetty or breakwater to serve under severe sea conditions. If based on sand as most jetties are, settlement from weight, currents, and wave action will result in displacement of surface capping components whether of stone or concrete special shapes. If they are nested, the sea, entering any break in the surface due to settlement, allows attack on the core materials, and disintegration results. It is, accordingly, considered much better on new work to use random placing to considerable thickness, so that a readjustment of the components will retain the integrity of the surface and a protective cover for the core. New jetties on a sand bottom should, accordingly, be left to settle for a long period. Then it may be found practical and economical to retop and place capping stones or other nested units on a rebuilt surface slope to secure a more stable structure. Under adverse conditions, it may be necessary to allow two or three periods of settlement and retopping after extended sea attack, before the submerged part of the structure attains stability. On a rock or other solid foundation, the problem of settlement is not present.

As indicated by the author, the end of a breakwater or jetty that is generally subject to the action of heavy cross seas is the most difficult part to maintain. At this point, heavy seas run and break across the end slope at high velocity. And, because there is little or no backing, the slope components are moved off to water on the lee side.

A heavy concrete terminal block, built in place at low water level, similar to that shown in Fig. 20, is considered the best solution for the superstructure. A long, flat, well consolidated, and settled slope below low water should be provided, however, beyond the terminal block.

Through experiences with the end of the Columbia south jetty, it was found that the cross seas unravelled the end of the heavy enrockment above low water, at a rate of about 300 ft a year, and all rock was swept off down to low water level. An asphalt and sand mastic was then tried as a binder, about 12,000 tons at 400°+ temperature being poured into the voids in the rock above the water and covering a distance of several hundred feet at the end. This was not successful, and, in fact, the sea cut the end back at practically the same rate as before the asphaltic mastic was placed.

In regard to the asphaltic mastic it must be said, however, that it was not possible to penetrate much below the high tide water level. There, accordingly, was a zone of weakness left at the most vulnerable level, between low and high water.

The heavy concrete terminal block (Fig. 20) was placed in 1940, and is still (1960) in place.

General Observations.—Jetties, to serve their basic purpose, need not be higher than high tide level, and indeed 1/2 tide jetties will, when maintained, serve fairly well as current-control structures. The purposes in building them much higher are (1) to make construction more practicable in rough waters in which a floating plant cannot be used, (2) to serve also as a breakwater, and (3) to provide a structure that will have a longer useful life before retopping becomes necessary. Adjacent to the beach, in the surf zone, jetties should be high enough to prevent heavy surf and currents from carrying sand across to the channel side.

Tidal ranges have to be taken into account. On the Oregon coast, tides range from about 7 ft to 10 ft.

The scientific approach referred to by the author is as desirable in breakwater design, as it is in the design of any structure. However, it is found, in many situations in which deep water and heavy sea conditions will not allow the use of floating equipment with any consistency, that the theoretical and desirable flatter slopes are often impracticable of placement with land equipment normally available for such work. Dependence is then placed in the action of the sea over a long period of time to work the materials into a flatter, reasonably stable slope.

Also, under adverse sea conditions, it is generally not practicable to place large stone or other components in keyed or carefully nested positions down near the low water level, where it generally is unsafe for workmen. And, this is the zone in which the superstructure is most vulnerable to sea attack.

Considering the necessary weights of components for high wave conditions as determined from various empirical formulas, it is found that the units are much larger than can be handled in the field; still, experience shows they may not be large enough to withstand heavy wave action without shifting, as previously related. There is an inherent conflict between what theory may show to be required and what it is physically and economically practical to build, except in the case of the lower range of wave heights.

The failure of a well constructed jetty or breakwater does not take place all at once, as may be the case in an underdesigned timber, concrete or steel structure. It does not occur in a single day or storm, but is a progressive process. Only minor damage may be noted after the first storm season.

After a break starts in the armor, disintegration proceeds at an accelerating rate, particularly if a large part of the core is composed of small units. For this reason the so-called C material should be kept to a minimum, and A and B stone used for a greater part of the core. Disintegration will thus be retarded. More attention should be given to design of the interior of jetties and breakwaters for service under heavy sea conditions.

Because of the time required for substantial deterioration of a well-constructed jetty or breakwater to take place, the duration of run in model studies at the time scale of the model, should stimulate the total duration of heavy sea conditions that is estimated to occur at the site in a long period of time. At Columbia River, it is estimated that heavy seas prevail for about 30 to 40 months in a period of 20 yr.

The author's statement that stone structures may, in some cases, be more economical than other types is very true, especially where good stone can be secured at reasonable cost. Then it is also to be remembered that, because the required weight of the components varies inversely as $(S-1)^3$, good stone has a considerable advantage over concrete.

The economics of breakwater or jetty construction, from the standpoint of overall cost over a period of years, is a matter for careful consideration by the engineer in designing the structure. In the writer's opinion, and from experience, it appears that a structure that is effective for a period of 25 yr to 30 yr, under severe sea conditions, before rehabilitation or retopping is required, may be considered as successful design. To attempt to build a structure to serve for a much longer period, under such conditions, will generally involve initial costs higher than can be justified.

J. E. DEIGNAN,[6] F. ASCE.—The paper contains an excellent summary of breakwater construction and experience in the Hawaiian Islands. The author should be complimented for his fine presentation.

The author correctly emphasizes the need for additional design criteria pertaining to (a) the design of the heads of breakwaters, (b) the design of structures subjected to severe overtopping, (c) the design of berms for stability against breaking waves, and (d) the maximum effective distance of breaking waves from the point of breaking. In regard to item (b), need for additional data on waves generated in a harbor due to overtopping is needed and should be added to the list.

The author discusses stability of keyed rubble armor layers where the armor stones are fitted together, and states, for breakwaters so constructed in the Hawaiian Islands, "This greatly increased the stability of the structures." It would appear that the coefficient of friction would be increased by the keyed construction pointing to the importance of friction between rubble or any artificial breakwater units.

It is stated that the wave attack from Hurricane Dot, although it reached design height, could not be considered as a conclusive test of the structure at Nawiliwili. In the next sentence it is stated that more severe attack would be produced by longer period waves. As Hudson's equation for the weight of breakwater components does not include the period of the attacking wave it is not understood how a change in period (with no increase in height or direction) would produce more severe wave attack.

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[6] Chf., Engrg. Div., Corps of Engineers, San Francisco Dist., San Francisco, Calif.

PREDICTION OF MAXIMUM PRACTICAL BERTH OCCUPANCY[a]

Discussion by Donald C. Hill

DONALD C. HILL.[9]—In the plans for development of general cargo terminal facilities at Hampton Roads, Virginia, the problem of financing is paramount. The Virginia State Ports Authority can finance the construction of new facilities only if they are able to demonstrate, to the satisfaction of the presumptive tenant, that the cargo potential to the terminal is of sufficient volume to justify the rental demanded. Obviously, then, it becomes essential that they be able to predict with a reasonable degree of accuracy the expected berth occupancy of the facility.

Messrs. Fratar, Goodman, and Brant base their prediction formula on the Poisson Law of Random Distribution. The Poisson Law, however, as would be true of any law of probability, is applicable only when all controllable factors have been eliminated, and the law applied only to those factors that may be characterized as pure chance. It follows, therefore, that the Poisson Law can be used for the prediction of berth occupancy only if applied to a terminal in which all available berths are equally attractive to all users. This law is not applicable unless all berths within the terminal under consideration are equally modern, equally maintained, and with equal access to all forms of transportation on both land and water sides.

At Hampton Roads specifically, the Poisson Law will give a "goodness of fit" of a very low order if we attempt to apply this formula to the entire general cargo terminal complex. It would be applicable only if applied to a single group of terminals served by a single carrier, such as the Chesapeake and Ohio facilities at Newport News, or the Norfolk and Western facilities at Lamberts Point in Norfolk, Virginia, or the Atlantic Coast Line facilities at Pinners Point in Portsmouth, Virginia. Even then, an unknown constant should be introduced to compensate for the variable age of the individual piers within the group.

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PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. The publication sponsorship is indicated by symbols at the end of each Paper Number, the symbols standing for Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (ID), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbol letter (PP). For titles and author coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 40 (January 1970) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the numbers after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1770 is identified as 2270(2:70) which indicates that the paper is contained in the 70th issue of the Journal of the Structural Division during 1970.

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† Publication of several papers delayed by disbursal.

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